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JOHN C. TRAUTWINE, JR., *Secretary*, 257 S. FOURTH STREET, PHILADELPHIA.

CONTENTS.

VOL. XVII, July-December, 1896.

For alphabetical index, see page v.

No. 5. NOVEMBER.

No. 6. DECEMBER.

INDEX.

VOL. XVII, July-December, 1896.

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PRINCIPLES GOVERNING THE DESIGN OF FOUNDATIONS FOR TALL BUILDINGS.

BY RANDELL HUNT, MEMBER OF THE TECHNICAL SOCIETY OF THE PACIFIC COAST.

[Read before the Society, April 3, 1896.*]

THE modern tall building is in many respects a good personification of our national character. Utility first and foremost, adaptability to immediate surroundings, and a capacity for acquiring the "almighty dollar" with ease and certainty.

It is "all things to all men"—convenient to commerce, comfortable and healthy. Professional and business life within its walls is not worried by the little annoyances which in ordinary surroundings are apt to detract from the more serious pursuit of material happiness.

ORIGINATED IN CHICAGO.—It is not surprising then that Chicago, most typical of all the cities of our national characteristics, should have been the place where these wonderful structures have received their greatest development. Nor was it cause for astonishment when it was discovered that the soft, muddy subsoil of this city precluded the use of the recognized methods of founding ordinary buildings; that a new and distinctly original solution was applied to give a safe and permanent support to the towering structures.

It can hardly be said that this result was achieved at once for any one building, for the true underlying principles of all foundation work were rather forced upon the builders and designers by a series of mishaps, which demonstrated in a most practical manner that scientific principles are the only true and safe ones to follow.

* Manuscript received May 4, 1896.—*Secretary, Ass'n of Eng. Socs.*

Necessity of providing for settlement under certain conditions.—If a particular soil will support but a limited weight without compression and settlement, then one must make suitable provision for such change in the position of the base, which of necessity must occur in any structure founded upon it, and exerting a pressure beyond that amount it will carry without any yielding.

In the case of a building certain principles of construction have been recognized as necessary—when it is founded upon a compressible earth—to prevent unsightly cracks and sometimes dangerous results from occurring.

To keep the area of the base of the building so large that the pressures transmitted to the earth will cause no settlement whatever is often regarded as impracticable, and in many localities it has been found that after a certain limited compression of the soil has taken place no further settlement need be apprehended.

This at least is, and has been, the argument in some places where many of the largest and most costly of our modern buildings are being erected.

Method of independent piers.—The method of founding large buildings upon independent piers is one now so common and so well understood by engineers and architects as to hardly call for any particular explanation here. It is simply a recognition of the well-known fact that if a beam is acted upon by two forces at or near its ends it tends to assume a curved form due to the unequal moments of the pressures transmitted to the beam from the reactions of the ground upon which it rests.

Fig. 1.

If this beam, as in the case of masonry connecting walls, is weakened by numerous openings, as windows or doors, one above the other, or by any other means, so as not to have sufficient strength in itself to transmit the pressures on its ends to the ground beneath throughout its entire length without deflection from reactions, then it will bend or crack on the lines of least resistance.

Uneven foundation pressures immaterial on rock.—In solid soils, or upon rock, or upon any perfectly unyielding foundation, it is immaterial, of course, how unevenly the pressures may be transmitted to it, and unless there is overloading to the crushing point, the principle of independent piers is of no practical use, excepting from an economical and perhaps convenient point of view.

Damage from settlement.—The upward reaction between the points of great pressure in a compressible soil usually results in a building being damaged by cracks extending from the base to the roof and following the line of windows from story to story.

Cooper Institute.—To illustrate briefly this common cause of failure in buildings, I have chosen as an example of the results which will happen from neglect of the principles just explained the Cooper Institute of New York.

This building was founded in 1853 upon piers carried down 22 feet below the sidewalk, and resting upon a continuous masonry footing course 1 foot 4 inches thick. In 1885 the settlement and consequent cracking of the walls had become so great that it was considered dangerous, and extensive reconstruction of the foundations was entered upon.

An examination of the building shows at a glance that the chief weight of the walls is carried down the piers "A" and "B" to the foundations, and that the pier "C" between these two carries but a small weight in comparison. From bad proportioning of the footings of these main piers only a limited area was capable of receiving the full pressures, and as much as six tons per square foot was thrown on the foundation soil. Failure occurred by the continuous stone footing cracking across, and the main piers were shoved down into the overloaded earth, tipping up the outer edges of the stones directly under them, which were not continuous but jointed in the center. At the same time the intermediate pier, with its light loading, remained without settlement, and as the piers on either side sank the upward reaction was sufficient to cause a "vertical fraction at each side of every window from the third story down."

If all the footings of the piers had been properly proportioned, so as to have exerted a uniform pressure per square foot on the soil, and this had been well within its safe supporting power, there would have been no accident. But as this is at best a difficult thing to always secure, viz., a soil which does not compress at least a small amount, even with light loading, other methods of supporting the intermediate pier could have been adopted if unequal settlement was feared.

Characteristic construction of modern high buildings.—The modern high building consists, in most recent examples, of a steel skeleton frame from the foundations to the roof, in itself carrying all the weight, from story to story, of the masonry walls, partitions and floors. These walls are reduced to the least dimensions—to sustain themselves only—being merely "curtain walls" in most cases.

The basements are usually required to have plenty of light and as much or more openings than the stories above. Therefore the method

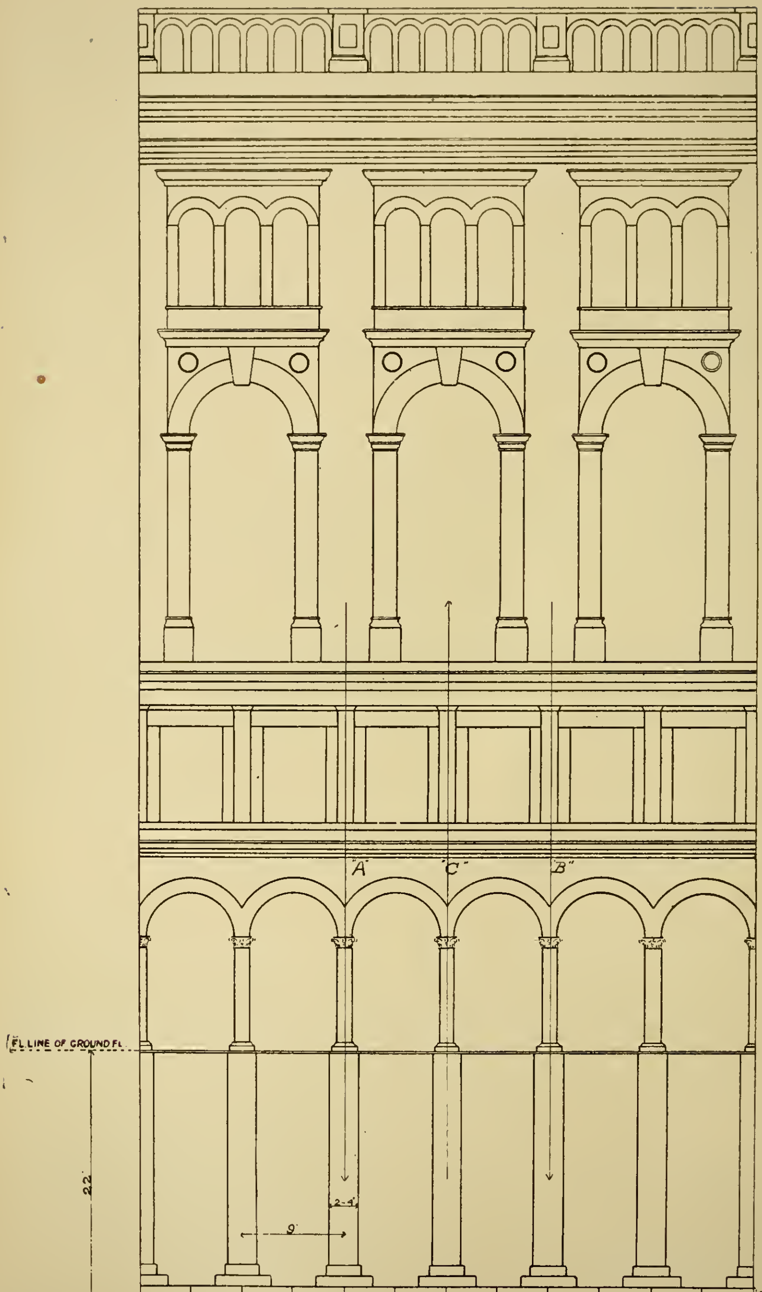

FIG. 2.—COOPER INSTITUTE, NEW YORK.

of placing each column, or pair of columns, upon its own base, separated and independent from the next set, becomes a very convenient method of economizing space for light and available area for business purposes.

Masonry piers under columns.—If there is plenty of good, hard ground underneath the level of the basement floor, the area of base necessary to be placed under each steel column could be easily attained by excavating to sufficient depth, so that a masonry pedestal could be built, with only the usual safe slopes or offsets from the bedplate to the ground.

If the foundation soil was very hard and incompressible—capable of carrying large loads, say of five to six tons per square foot—then the ground area required for the foundation piers would be small, and consequently they need be of but limited depth in the ground. But if, on the other hand, the supporting power of the soil is very small and liable to compression, reducing the unit of safe weight which can be placed upon it to from one to two tons per square foot, then it becomes necessary to have a foundation base of large area. This can only be accomplished by making the masonry pier of considerable depth down in the ground, so as not to exceed the proper safe slope or projection of the footing courses.

Advantages and disadvantages of this construction.—There is no objection which can be urged to this method of securing a foundation if the soil is a suitable one for the practical carrying out of the construction.

In fact it has many distinct advantages over the usual shallow steel beam and concrete foundation.

Unfortunately the conditions of the foundation sites in many of our large cities, as well as extraneous conditions, such as neighboring buildings, ground, water, etc., make this method at times more or less impracticable.

Founding upon a bed of firm soil overlying softer material.—In Chicago, for instance, the peculiarities of the ground formation make it entirely out of the question to found in the above manner, for almost without exception the records of the borings of the foundation sites of the large buildings show a depth of only from twelve to fifteen feet of a very moderately firm soil overlying a much softer clay subsoil of considerable depth.

The use of this top crust of firm ground, without in any way cutting down into it, has been the object sought by most of the architects and engineers in founding their large structures.

Origin of concrete and steel beam foundations.—Hence, as is usually the case, necessity became the mother of invention, and

steel beams in shallow piles, placed in tiers at right angles to one another, took the place of more bulky masonry, and attained the same purpose without requiring but very limited excavations.

Adopted by force of circumstances for peculiar conditions, yet the method of construction has shown itself to be more or less well adapted for other localities where these surroundings may not exist.

A great change in the relative cost of materials has also been another factor in the use of these foundations. Steel beams are almost 100 per cent. cheaper in cost to-day than fifteen years ago, and, in addition, their properties and use are better understood.

Mr. Bauman, an engineer of Chicago, called particular attention some years ago to the method of independent piers for foundations in compressible soils, and announced it as a scientific principle of construction, which was a necessity in soils such as in Chicago. He simply explained a very ancient foundation method in use since the times of of the Goths. It is certain, however, that the practice has followed on the lines indicated by him with more or less success.

Calculation of weights supported by independent piers.—If the lines of pressures on the ground area under one pier overlap those of a contiguous one it becomes necessary to make a single base for both piers. In this way there has developed a growing tendency in the later examples of large buildings to lessen the number of independent foundations by grouping several columns upon one base.

Undoubtedly independent piers, if very carefully proportioned to the exact weights carried upon their bases, as well as taking note of the friction upon their perimeters, offer a proper method of securing a foundation for tall buildings.

There are, however, certain principles not so easy to calculate in definite terms, which introduce more or less difficulty in arriving at a correct area to give the bases of columns transferring very unequal weights.

It is a known fact that large areas of soft soil will not support the same weight per unit of surface as more limited areas of the same soil.

It becomes necessary in designing the bearing area of the base of the foundations to take into consideration this fact if one is to feel perfectly certain of an equal settlement of all the piers.

An equal allotment of weight to be supported per square foot of ground area, under small piers as well as under much larger ones, will certainly result in an unevenness of settlement due to this principle just enunciated.

Chicago practice is the result of experiment.—Before the era of extremely tall buildings—some twelve years ago—various methods of founding in that peculiar soil had been tried. The more common

practice of to-day is largely the result of actual experiment and successful precedent. Still one cannot but be impressed with the fact that there is not a uniformity of opinion that correct methods have yet been adopted. One is startled to know that buildings which have cost one and a quarter to one and a half millions of dollars, are expected to, and do, settle five to six inches during the first year or two of construction.*

The floating of such buildings upon a crust of soil but twelve to fifteen feet thick, overlying a softer watery clay, shows a reliance upon future stability which is sublime, and entirely Chicagoan in its assurance.

INVESTIGATION OF THE DIRECTION OF THE GROUND PRESSURES.—If reliance is placed upon the strength of a top layer of soil which overlies a weaker material, then an investigation should be made to determine if the foundation pressures are distributed over a sufficient area of the lower soil to be within its safe bearing capacity.

It was at one time thought that the angle which the direction of the pressures through the ground made with the vertical was equal to the natural angle of repose of the material.

Experiments—in sand particularly—would seem to indicate that this is not correct, and that the angle is really about one-half the slope angle of the earth.†

Let the Fig. 3 represent the foundation of a column in a building resting upon sand. The angle of repose of ordinary moist sand is about 40°, in which case the angle of pressures becomes 20°; therefore the dimensions of the ground area which receives the original foundation pressures can be easily investigated at any depth.

Supposing a strata of soft clay, or other material, to exist, then by drawing to sufficiently large scale the figure herewith shown, one can easily measure off the dimensions receiving the pressures and see if the load per unit of area is within the safe pressures which is permissible on soft clay.

Let $\Phi =$ the angle made by the direction of the pressures, and this is equal to one-half the natural angle of repose of the soil.

Then the width of base b', which receives the foundation pressures at a depth h, is:

$$b' = b + 2\,h \tan \Phi,$$

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* The Monadnock Building, nearly 200 feet high, with a pressure on the foundation soil of 3,750 pounds per square foot, settled very uniformly five inches.

Old Colony Building, 215 feet high, with 3,220 pounds per square foot on foundations, in nine months settled from $4\frac{3}{16}$ to $5\frac{1}{8}$ inches. Both these buildings are considered successful examples of Chicago practice.—*Engineering News*, October 13, 1892.

† Handb. d. Ingen. Wissensch., Bd. II, p. 196. Wochenbl. d. Ver. deutsch. Ingen., 1882, p. 53. Experiments by Brenneke and Forchheimer.

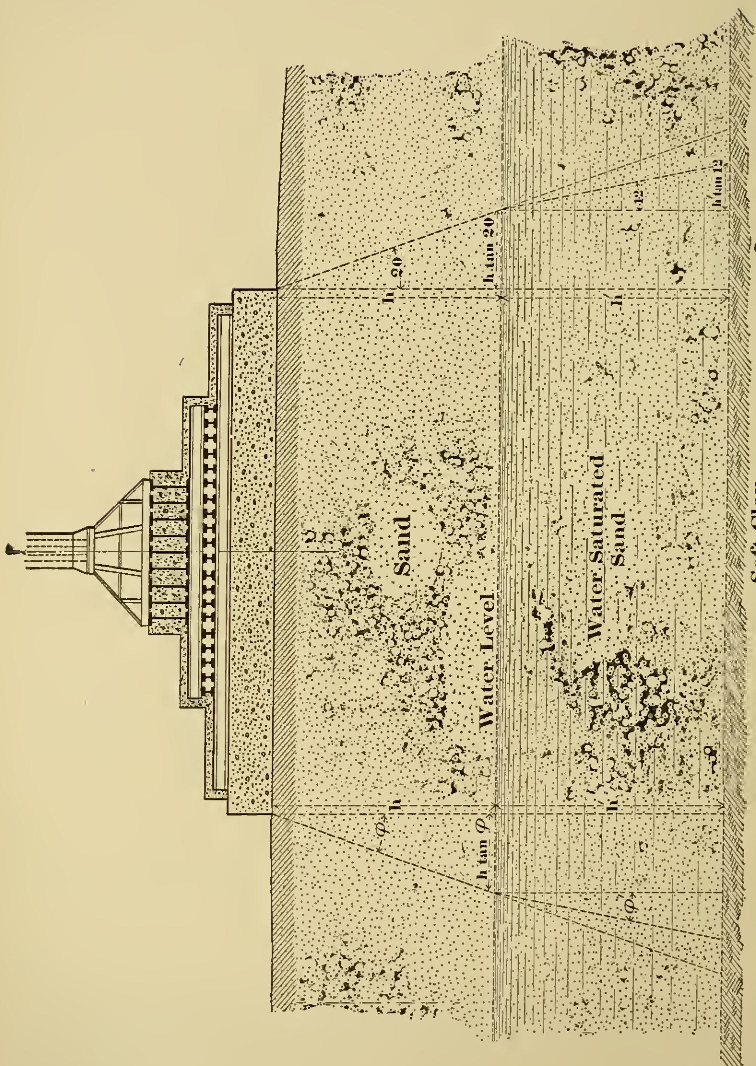
Sand
Water Level
Water Saturated
Sand
Soft Clay
h tan 20
h tan 12
h tan φ

Fig. 3.

in which $b =$ the width of base of the foundation pier at the surface. At a depth of $h + h'$ the width of base b_2 is equal to

$$b_2 = b + 2h \tan \Phi + 2h' \tan \Phi',$$

in which Φ' represents a change in the angle of pressure direction due to a water-soaked material, which has a much smaller angle of repose than the same sand above the water level.

If the area of the soft clay at this depth is not sufficient to give a value per unit of surface less than its safe supporting power, then the dimensions of the base of the column must be increased.

In addition, one must take into consideration the weight of the earth itself which lies between the base of the foundation and the strata of the soil considered.

Narrow limits of pressures in water-bearing ground.—The natural angle of repose of water-soaked soil becomes very much less than when dry or simply moist.

The very common condition in the foundation of a building is that the level of the ground water is within a few feet of the base; therefore, although the angle of the pressures of the foundation may commence, when the ground is dry, to extend itself over considerable area, yet, as soon as the water level is reached, this angle becomes very much less.

For sand, the natural slope, when water-saturated, is about 24°, and this would limit the pressure directions to only 12°, as shown in Fig. 3.

In reality, the natural slope of repose of the earth under a foundation base is not capable of too close an analysis. It is undoubtedly affected by depth, degree of moisture and the lack of uniformity in the character of the soil. Yet an investigation can usually be carried out on the lines which have been suggested, so as to leave no doubt as to the limiting thickness which a top crust of soil ought to have, to properly distribute any weight placed upon it, to a weaker subsoil.

Earthquake effects.—Vibrations are more or less injurious to all structures, and good construction seeks to reduce them to a minimum. The relationship of the foundation of a building to its superstructure is of much importance in this respect, and in any country subject to earthquake shocks due regard should be paid to avoiding their effects.

Careful investigation in Japan, extending over many years' observations, as well as in other earthquake countries, shows that unless the locality is situated directly over the center of the disturbance there is seldom any damage to well-constructed substructure work.

In certain portions of South America so well is this understood that the lower stories of the buildings, while of heavy masonry, or adobe, will have pliable basket-like construction in the upper stories.

The movement of an earthquake is vibratory, and in those parts of the

United States in which they have occurred the amplitude of the vibrations is comparatively small, so much so that well-constructed masonry in considerable mass in the ground is capable of taking up the oscillations without damage.

The greatest intensity of the shock and the amplitude of movement is at the beginning, rapidly diminishing during its duration, much like the vibration of a tuning fork, or like sound-waves.

Therefore the oscillations of a foundation of a structure will be the same, and no more, than those of the ground in which it is built. The building above, however, may become subject to a cumulative vibration derived from the oscillating base, and this is the usual cause of disaster.

This is due to the fact that the vibrational period of the building itself, caused by the first shock, becomes a multiple of the earth vibrations, and the amplitude of them is thus increased, though in reality those of the earth are diminishing.

Such oscillations might throw additional strain on the foundations of a building, and from the existence of different vibrational periods in one part of a structure over another—if the building rested upon inde pendent piers—one or more of them might be called upon to carry greater pressure to the soil than others.

Steel buildings safe in earthquakes.—Before the days of recent steel skeleton structures, independent authorities in different portions of world, who were seeking a proper design for buildings which should be earthquake proof, recommended iron frame construction.

The idea being the building should be tied and braced together in all parts; that they should be light in weight as well as strong; that any vibration should be as a whole, and not greater in one part than in another.

The superstructure of the modern steel frame building complies with all these conditions, although no thought of earthquakes had anything to do with the general growth of the design.

Not so with the foundations however.

Movement of buildings during earthquakes.—The moment of inertia of a heavy overhanging roof, or top of a tall building, seeks to keep it at rest, and if the base is set in motion by a sudden shock, great forces occur tending to cause rupture between the foundation and the superstructure.

The building being strong enough to resist rupture at this moment of time, the great roof now moves forward, and the energy of its movement may be increased by coinciding with the vibration of the ground. It resists any sudden checking of its motion only by causing again strains to occur of the greatest magnitude.

How the foundations should be designed.—It appears to me that in a building of great height, in which, of course, the amplitude

of the vibrations at the top is extremely liable to be greater than at the base, that this base in all its integral parts should move as a whole, and no part of the foundation should be able to transmit an unequal movement to the superstructure.

Considerable mass and weight in the foundation will in itself take up and destroy part of the movement of the earth before transmitting it to a building upon it.

Shallow foundations not proper for earthquake vibrations.— The method of independent piers, as now built in common practice, makes use of small mass and weight, and would transfer any earth movement in the quickest and most direct manner to the steel frame resting upon them.

Prof. Milne found at the college at Tokio, Japan, a difference in the intensities of the earth movements during an earthquake, even over a very limited plot of ground. For this reason it is certainly better to make a single foundation for a building, or one which must move as a whole. It has also been determined that there is less vibration at a depth of some feet in the ground than on the immediate surface.

Of course, in buildings covering very large areas of ground this becomes well nigh impracticable, or unduly expensive, but a continuity in the foundations should be sought to as great an extent as possible.

Foundations should be designed to act as a whole.— If due regard is paid to the principle mentioned in the first part of this paper—to avoid throwing upward reactions upon connecting walls, or of transmitting shearing and transverse strains upon insufficient masonry construction—foundations can be so designed as to act as a whole, and without causing deformations or cracks in the superstructure.

Solid masonry often cracked by earthquakes.— In the earthquake which occurred a few years since in the Vaca Valley of California it was observed that less damage was apparently done to some of the older and more "flimsily" constructed buildings than to those of more firm and rigid masonry. This was due to the fact that the old buildings were loose-jointed, and simply separated and pulled apart in many places without uniform vibration as a whole. On the other hand, the firm brick walls cracked throughout their entire lengths.

This is in accordance with Prof. Milne's investigations in Japan. He says: "An important point, which constructors should keep before them, is to avoid coupling together two parts of a building having different vibration periods, or else to couple them together so securely that they shall move as a whole."* The trouble with the Vaca Valley buildings was, they had sufficient strength to gather great vibrations, but not enough to resist final rupture.

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* Inst. of C. E., Vol. C.

Proportions to resist earthquakes. — Theoretically, the weight of a high building should decrease uniformly from the roof to the foundation. The weight of one story, as concentrated mostly in its floor system and in its exterior walls about the floors, should not be carried upon too slender piers to the story below.

Unfortunately the modern demand for light and space makes a very undesirable condition of affairs in this respect; in the first one or two stories above the foundation very often the ground floor is given up to shops, and nearly all the space which should be in walls or heavy masonry piers is converted into large windows and openings. The entire building over this floor is generally carried upon iron pillars. The vibration of the massive structure above them can only be transmitted to the foundation by means of these small columns, throwing a duty upon them which is most tremendous; and, in fact, they are unsuited to taking up these vibrations and transmitting them to the foundations and the ground.

The building does not vibrate as a whole, and cannot do so with this method of construction. And particularly in the modern structures of great height should attention be paid to this principle of providing mass and weight in the base, with the least possible amount in the top of the building. Between the roof and the foundation both mass and weight should be gradually proportioned without such open construction as to permit of the independent vibrations of different parts of the building.

Steel beams in concrete.—Concrete and masonry have not, as a rule, much transverse shearing or tensional strength. When used in foundation work it should be the aim to so proportion their dimensions and positions that they will be subject to compression only. Steel beams introduced into concrete do away with the deficient strength of the concrete alone and renders it safe for transverse strains. For this purpose this construction becomes most valuable in foundations, not the least of which is that the exact dimensions needed are susceptible of accurate calculations.

Magnitude and weight.—It is not wise, however, in many cases, especially when earthquake vibrations are liable, to make the foundations too shallow, simply because the steel beams in themselves may have sufficient strength to take the strains that come upon them, for the reasons which we have explained before, of providing a base of magnitude and weight to take up the vibrations transmitted through it.

In the first part of this paper I have mentioned the fact that there is no objection to foundations of masonry alone without depending upon an interior steel stiffening. What I mean by this is that such a foundation, proportioned mostly for compressive strains, requires considerable depth in order to secure sufficient area of base, and hence acquires large mass and weight. The considerations which have just been shown are

among the chief ones to prevent undue transmission of vibrations to a superstructure built upon them and to give great rigidity to the structure as a whole.

MONOLITHIC FOUNDATIONS.—I find numerous examples in German cities of successful monolithic concrete foundations under heavy buildings. But in all such cases the great thickness of the concrete is evidently relied upon to give sufficient strength to the base to resist uneven reactions. The Nicolas Church in Hamburg rests on a bed of concrete 8 feet thick, while under the tower this thickness is increased to 11½ feet. And other buildings are recorded with depths of from 5 to 6 feet.

In contrast to these the monolithic foundations of Chicago have generally failed. The City Hall settled very unequally, as much as 14 inches; but the bed of concrete under it was about three feet thick, and this appears to have been as great a thickness as was used in any of the other buildings upon such foundations.

A monolithic concrete foundation stiffened with heavy steel beams at right angles to one another, and not too shallow, makes an ideal system of construction, provided correct dimensions are given to the base.

FOUNDATIONS OF THE "CALL" BUILDING, SAN FRANCISCO.—The building now being built by Mr. Claus Spreckels, in San Francisco, and commonly known as the "Call" Building, rests upon a bed of sand. It is almost square, being 70 feet by 75 feet in plain dimensions. The foundation consists of a layer of concrete 24 inches thick, then a tier of 15-inch steel beams spaced 18 inches centers, and then another at right angles to this of the same dimensions, all void space being filled with concrete. On top of this solid platform rest 20-inch beams grouped together under the columns as shown.

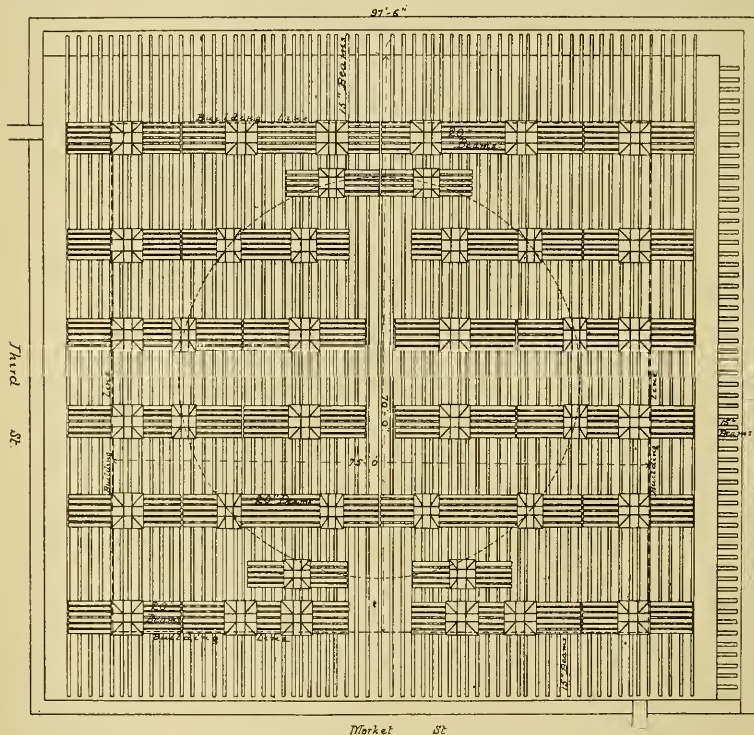
On the sides of the lot adjoining other buildings the steel beam and concrete platform is extended, so as to be used in supporting them. All beams are spliced, so as to be continuous from end to end. The outside columns are anchored to the foundation with two anchor bars of steel 1½ inches by 8 inches, which are fastened to the under side of the lowest tier of beams and extend up and into the columns themselves, to which they are riveted. Messrs. Reid Bros. are the architects for this building.

PILE FOUNDATIONS.—To us on the Pacific Coast it appears strange, in investigating the foundations of large buildings in other parts of the country, to notice the avoidance of piles as a means of founding.

Both in Chicago and New York the underlying hard pan and rock appear to be within the reach of long piles. At Chicago, firm clays appear from 45 to 50 feet from the surface, and the rock about 80 feet, while in New York the rock generally occurs in less than fifty feet.

The shock of driving piles next to other buildings undoubtedly has been one cause which has in some cases precluded their use; but it would appear that in Chicago it has been largely due to a distrust occasioned by the failure of several buildings founded on piles a number of years since. By the advice of several prominent engineers of late years a very few of the most recent buildings have been placed upon piles.

Piles make firm foundation.—Under the conditions of piles



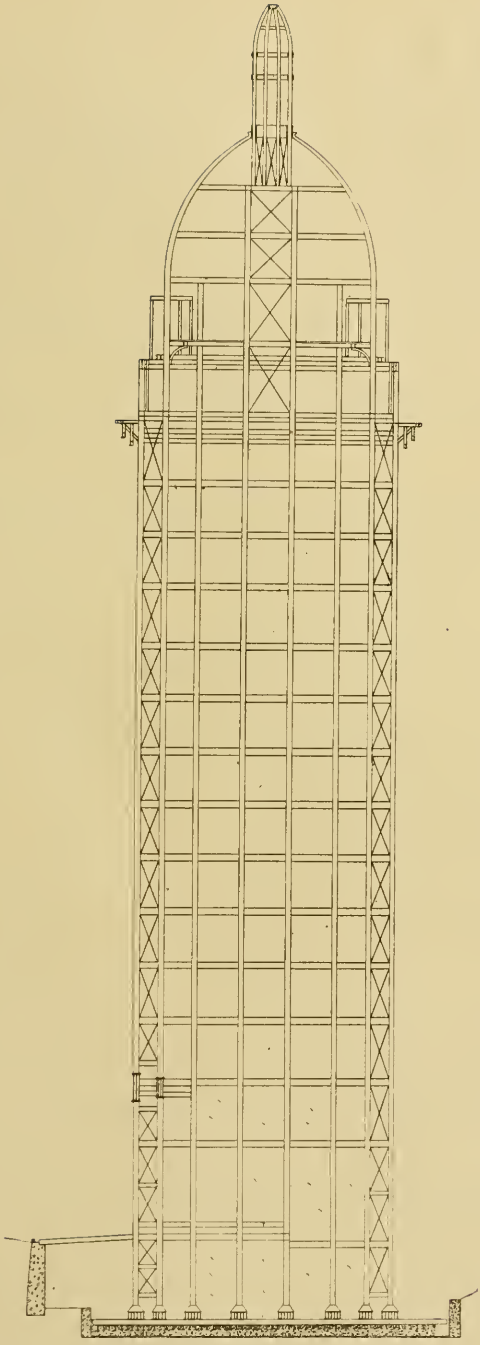
Plan of Foundation of the "Call" Building.

Fig. 4.

being driven into a firm soil there is no better foundation, provided they are kept continually wet, and hence safe from decay.

When they depend entirely upon a small surface friction on their sides, as in our own shore mud and clays, we know even then that when driven to great depth they are safe for moderately heavy loads.

I desire to carefully avoid any discussion here of the very broad subject of the supporting power of piles, beyond the general statement that it is but a rare situation and condition of things when full and



Section of the "Call" Building, San Francisco.

Fig. 5.

ample supporting strength cannot be secured by a properly driven pile foundation.

Should be driven deep.—If piles driven thirty or forty feet do not show sufficient resistance during driving, and the character of the soil is such that one does not care to risk its remaining with the same supporting power after a lapse of years, I can see no reason for stopping at this depth.

This is, and always will be, one of the chief causes of pile foundation failures, viz., the inability to recognize the proper depth to which piles should be driven. As I have said before, it is a very broad subject, and has many most interesting and abstruse matters relating to it, which would require an even longer paper than this one to make clear.

Grillages.—A grillage of timber on the heads of piles makes a most efficient base upon which to found a structure. In distinction from the very common method employed—largely in Europe—of placing concrete upon and around the heads of piles, it is the more usual American practice to build upon a timber grillage.

I think it is obvious, for the reasons stated before, that transverse breaking strains should be avoided in masonry of any kind ; also because of a ready means of tying and bracing together in various directions all the piles of a foundation, and thus causing them to act as a unit for stresses of all kinds that the timber grillage is much the more preferable method of construction.

Cantilever construction.— A not uncommon condition of affairs, in designing the foundations for a tall building, often occurs in which it is necessary to keep entirely within the lot line and the outer line of the walls of the building with the substructure work. Under such circumstances it is difficult to avoid throwing undue strain on the outer edge of the footings. If the soil is a compressible one it is of the utmost importance that the center of the ground areas and that of the pressures transmitted to them should be concentric. In the case of buildings founded directly upon the soil this is often very difficult to manage satisfactorily, and in fact, under any method of founding, it is undesirable. In New York this problem has been met, in several notable instances, by the device of constructing the building upon the great steel cantilever beams, which overhang from tubular piers founded upon the solid rock, and placed entirely within lot lines.

By this means the center of pressure from the weight of the building can be transferred to the center of the foundation areas.

Broad base most desirable.—It is always better construction in any structure to found on a broad base, and no matter how firm a foundation may be secured, a great, tall building, two hundred or more

feet in height, resting upon a base, the exterior edge of which encloses an area smaller than the plan of the structure itself, is not in the most desirable condition of stability.

Of course such designing is not the result of choice, but from necessity.

Foundation direct on sand.—Of late years it has become somewhat more common practice in the city of New York to found some of the great buildings directly upon the sand, which is a natural formation there, while in Chicago there is an undoubtedly growing tendency to found upon piles driven into the hard clay, or upon wells excavated down to the hard strata in the soil and then filled with concrete. The practice in both places, by a process of evolution, is simply approaching correct theoretical principles.

Sand generally safe.—Sand, provided the same is not liable to future disturbance by nearby excavations, or from flooding or other causes, is, as is very well known, a particularly good foundation, and permits of loading with considerable pressures per square foot and with a minimum of compression.

The late distinguished engineer, Alexander Holly, boldly founded an important structure upon a quicksand, but first took the precaution to permanently enclose it.

Factor of safety for supporting power of soils.—It is considered good practice, and entirely permissible, to strain up to 16,000 pounds per square inch on the steel beams used in tall building construction. This is about 50 per cent. of the elastic limit of the metal.

Why are not these conservative principles applied to the supporting power of soils in foundation work? Why is it more correct to load a soil which shows considerable compression and change in shape under a loading of, say 4,000 pounds per square foot, with a constant and unchanging pressure of from 3,000 to 3,500 pounds?

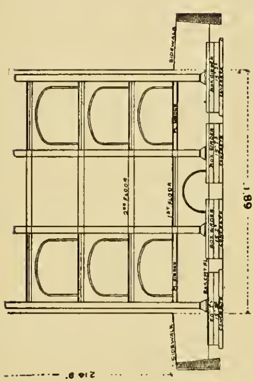
The author is satisfied that upon wet clay soils, or loam, or upon sand which is more or less impure, more moderate values of loading must be used than has been customary in the past in many of our greatest buildings.

Main principles.—The first and chief axiom in all foundation practice is to *know the exact character of the soil upon which the structure is to be built.*

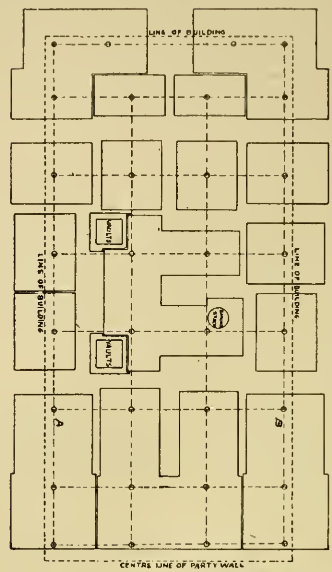
All other things become secondary to this great principle, and in fact resolve themselves into simple mechanical problems capable of definite solution.

All soils have certain safe supporting power, and are likewise susceptible of a definite amount of compression.

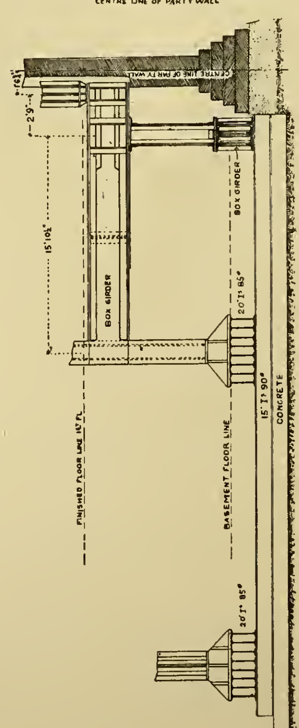
The loading of the same must be safe within the limits of such pressures as produce measurable settlement.



CROSS-SECTION ON LINE *A B*.



FOUNDATION PLAN,
OLD COLONY BUILDING, CHICAGO.



VERTICAL SECTION, SHOWING CANTILEVER.
OLD COLONY BUILDING, CHICAGO.

FIG. 6.

The investigation of the soil must be complete for considerable depths and particularly is this necessary for pile foundations.*

Having acquired a true knowledge of the physical characteristics of the building site, the foundation design must be such as to recognize the various principles commented upon above.

Architecture is a science and art largely based upon historical precedent, but the foundation of a modern high building is a question of engineering construction. Excepting that precedent furnishes information with regard to the strength of materials it is a dangerous rule to follow blindly.

DISCUSSION.

Mr. Wagoner.—I would like to ask Mr. Hunt if the soil under the "Call" Building is sand?

Mr. Hunt.—I have been informed that it has a uniform sand foundation to a depth of forty feet or more.

Mr. G. W. Percy.—I have enjoyed the paper very much, and especially what was said about building foundations in Chicago. Some twenty-four years ago, just after the great fire in Chicago, I was engaged on some of the heavy buildings there, and I have a special interest in Chicago foundations.

At that time the system of building on isolated piers was just beginning to be recognized as the proper thing. It was very seldom practiced. It was the common practice there among the architects, when any science at all was used in building, to make the maximum load about one and one-half tons to the square foot. I was engaged in the office of the leading architects in Chicago. After the great fire they did a great amount of work. In a year and a half they put up a mile and three-quarters of street frontage of buildings to be used for office purposes, and these buildings ranged from four to seven stories high. This firm was then an advocate of isolated foundations, although they did not practice it, except on some of the buildings in which they were very anxious there should be no cracks or any marked settlement. In the Kendall Building, which is an office building of fire-proof construction, some of the boys from our office were sent there every month to take levels through the building, to see if any one part was settling faster than the rest. We figured on its settling about two inches,

* A most important bridge foundation failed in Philadelphia, because in the driving of the piles they were left with their ends just about to penetrate an unsuspected soft strata of mud. If a careful boring had been made beforehand it is needless to say the piles would probably have been driven deeper and the accident averted.

and that the pressure would be one and a half tons to the square foot. When it was found that any column was not settling as fast as the rest, pig-iron was taken into the building and placed upon that column until it settled equally with the rest. A large amount of pig-iron was used to bring about this result.

Something of the same nature was practiced on the Auditorium Building, I believe. The building was of uniform height and weight up to the base of the tower where it emerged from the roof. The weight of the tower above the roof was between three and four thousand tons. The architects loaded the foundations of the tower with about three or four thousand tons of pig-iron, representing the excess of weight of tower above the roof, and kept that load on the foundations until the entire building was up to its roof-line. Then as the tower was carried on higher they would remove the pig-iron from the base, the object being that when the building was at the line of the roof the weight was equal on foundation, and equal settlements should have taken place, and they removed the pig-iron as they added brick and stonework. So when the tower was completed it had no more weight upon the foundation than when at the roof-line. That was quite successful, but not entirely so. Some considerable settling took place at the tower, which the architects explain by claiming that some five hundred or six hundred tons were added by changes and alterations made on the tower.

I think Mr. Hunt has explained the real causes why these very lofty buildings settle so much more than the one and a half inches which used to be recognized as the proper amount of settling.

It is evident that, given a thick layer of clay, such as they have in Chicago, of seven or eight or ten feet thick, and a softer layer of clay under this, the ordinary style of building of five, six, and seven stories high, and a pressure of one and a half or two tons to the square foot, as the case may be, distributed over the entire area of the building, it was not sufficient to cause the entire strata to bend or yield, and therefore that the settling of the building was just the compression of the harder clay. But with these enormously increased loads of the high buildings the whole body of this upper layer of hard clay settles, and it is the lower strata that is overloaded rather than the upper one. I think that is a proper solution of it, because it is quite certain that an ordinary building of six, seven and eight stories in height, loaded to two tons even to the square foot, does not settle more than two inches or thereabouts, while some of these heavy buildings have settled five or six inches. I think this increased load is transmitted to the softer strata below.

In regard to the question of piles in Chicago, the reason given in those days for not using piles was that there was something peculiar

about Chicago clay which was not adapted to their use. In most places where piles are driven in mud or clay, and even where it is quite soft, when a blow will drive the pile two or three inches, if you let it stand six months and then apply the same blow, it will not move it; but in Chicago it is the reverse, and you strike a pile after it has been left this length of time and it will go out of sight. It is claimed that after the fire, buildings on pile foundations settled considerably.

In this city we have a very good hard sand, and there are many buildings where the foundations are loaded to about four tons to the square foot without any perceptible settling, not perceptible enough to make any cracks or dislocations.

I would take some exceptions to Mr. Hunt's remarks, as I gathered from what he said that he did not consider it best to load foundations so near the yielding point, and that the soil should not be loaded more than one-half of its elastic limits. I do not see that the argument applies to foundations. In the case of masonry, or most any material employed in buildings, there is a possible deterioration going on; but in the case of foundations there can be no deterioration; the foundations of sand under a building do not deteriorate with age, and the load may be very near to the point to which some yielding would take place. But if loaded double or treble that load, no serious consequences would result, therefore I do not see why it is not prudent and safe to load a foundation to near its yielding limit.

I would also make one other suggestion. The members of the Technical Society know I have advocated the use of twisted rods with concrete in foundations, and that I have made some experiments in this line. I make a uniform foundation, a platform for the building to rest on. This method is a great economy in materials as compared to some others. Take such a case as this: A platform of concrete, say 4 feet thick, with a sufficient quantity of twisted rods placed both top and bottom of the concrete, and one-fourth of the amount of steel in this foundation, would be equal in strength in every particular to a foundation where steel beams are put in.

Prof. Marx.—It may be of interest to know that Mr. Sooysmith, who has probably carried out more important foundation work in this country than any other engineer, is to bring this subject out at the April meeting of the American Society of Civil Engineers. I just happened to glance through a little paper which details the point he expects to bring out. Mr. Sooysmith calls attention to the fact that pile foundations in a number of instances have not been satisfactory, owing to decay of the piles due to a change in the subsoil water levels. The supposition was that the subsoil water level would remain permanent, but this supposition was found to be wrong. The piles are alternately exposed to water and air, and under these conditions of course they rot away.

In the matter of sand foundations Mr. Sooysmith mentions the fact that in New York City, in the case of large buildings built upon sand, such a method of founding is rather dangerous; that sand is a good material when confined, but that oftentimes when a neighboring building is torn down and a new building put in its place, the sand gives way to some extent, causing a subsequent settling of the building. Then the responsibility for whatever injury may occur falls upon the man who has erected the new building. He therefore suggests—and I think it is the method he has carried out—that the foundations be carried down to the solid rock by the use of pneumatic caissons.

I mention this as showing that the subject of foundations is interesting, and is agitating engineers at the present time.

Prof. Wing.—A question that comes up in regard to foundations composed of steel and concrete is that of the durability of steel in concrete. While it has been accepted that steel will last, that it is indestructible in concrete, yet I think there has been no definite determination of that fact, and it is not definitely known that the material will last. Observation of structures that have stood for some time I think points to the fact that it will last.

The plan of building on a continuous floating foundation under a building is one that depends upon whether the pressures of the building are uniform over the whole area or not. This inequality of pressures in the case of the Chicago buildings is corrected by giving each column a bearing area under it proportioned to the load it carries. As I understand the matter, they carry out this design so completely that where two foundations of concrete meet they separate them by a board, so there shall be no communication between the two foundations, thus preventing the concrete under two columns from acting as a beam and producing eccentricity of pressure on the foundation bed.

Mr. Percy.—I want to state another interesting fact about the foundations of the Post Office in Chicago. The matter created considerable discussion at the time the foundations were put in. It has been referred to very frequently as a failure of continuous foundation. I had an opportunity of observing the way it was put in. If I remember, the concrete was supposed to be four feet thick over the entire area of that great building. I watched the process of putting it in through the cracks from time to time. There was a layer of six inches of broken stone laid down, and then cement grouting poured over it; then another layer of stone six inches thick, with cement grouting, and so on. The papers spoke of it as the most perfect method of laying concrete that had been devised. The result has been that the building has settled, and there are cracks in the building in some places two inches, in other places eight inches wide, and in some places perhaps more; there has even been a breaking of columns and beams throughout the building.

Prof. Soulé.—I would like to have Mr. Hunt, or any other gentleman in the room who knows, state what has been the amount of settling of the Appraisers' Building here in San Francisco. You remember its foundation is a thick layer of concrete. I suppose some of you saw that construction. I believe the concrete is spread over a larger area than the building stands on. I think the concrete is about six feet thick. Of course the building is a heavy brick structure. I have been told that the settlement has been considerable, but as far as I know it has been pretty uniform over the whole area. I fancy if the weight of the building itself had been evenly distributed over that concrete slab, that with the thickness and weight of the slab itself the building would have settled uniformly. There was a discussion at the time as to whether the Appraisers' Building should rest on piles, as does the Post Office Building next to it, or on a concrete slab. The latter plan was adopted, I believe, by the late Gen. Alexander and Col. Mendell.

With regard to pile foundations I think that sometimes after long periods of time unequal settlement occurs in the building, due to a different cause from those mentioned by Prof. Marx. In Venice a great many buildings stand on piles driven in the mud of the islands of the archipelago; they were put up from the year 800 to 1000, and so on to 1200, during the period of the glory of Venice. Some of the heaviest buildings there were erected, I think, about the year 1200 or 1300. From an examination of some of those I am satisfied that while unequal settlement in some of the buildings has caused them to lean over in a rather dangerous way, in fact in some places to threaten to fall down if unsupported by their neighbors, the settlement has been caused in some instances by unequal weights on different portions of the piling, while in others I am very sure (and I draw this conclusion from personal observation) that the settlement has been caused by the long continued weight upon the piles, causing an actual telescoping of the fibers one into the other; in other words, an actual shortening of the piles through the cellular spaces being diminished. I saw in some places evidences of the piles having been compressed in the direction of their length to a considerable degree.

In our pile foundations, if we expect them to endure heavy loads for hundreds of years, the question of a thorough equalizing of the load coming upon the piles would, I think, be quite an item in affecting the final stability of the structure.

Mr. Percy.—I would like to say one word in answer to Prof. Soulé's question. The foundation of the Appraisers' Building was put in before I came here, but when I arrived the building was not finished. I am acquainted with the superintendent of the cement work, and he told me what would indicate a dangerous condition of the foundation. There

was a great body of concrete laid over the entire area, and at the north end it was resting on rock, while the other end was resting on soft mud, which, of course, we all realize to be a very dangerous condition. I have watched the building to see if there were any cracks from settling. There are one or two small cracks on each side, showing some movement, but nothing of the extent that would be expected under such conditions as this superintendent described to me. I think the Appraisers' Building has stood very well, and there has been very little movement of the foundation.

In regard to pile foundations and the capping of piles I would like to hear some further discussion. Instead of simply capping the piles with timber I am strongly in favor of digging around the piles six inches below the tops of the piles and ramming concrete all around the piles. In this way we get a full bearing upon the piles, and in addition to that we get the bearing capacity of the soil, whatever it may be ; it binds the piles together as well as any grillage could do, and it is much cheaper. On the whole, I think the advantages are in favor of capping with concrete.

Within the past six months I have put up a building in this city alongside of a building resting on piles. I went below the foundations of the building, and I was curious to see what sort of bearing the building had upon the piles. I dug about the cappings, and I found places where I could push my rule in between the caps and the tops of the piles. The building had not been loaded heavily enough to bring the capping down. The piles were not cut off on an exact level line, and therefore the grillage did not rest on them properly. But in using concrete the way I have described, every square inch gets a bearing, and, to my mind, it is better than grillage.

Mr. Hunt.—The discussion has followed pretty near the lines I thought it would. I have been somewhat disappointed that nothing has been said in regard to the earthquake vibrations. There are only a few points that I would like to answer.

With regard to Mr. Percy's remarks about a foundation made of twisted steel rods in concrete, placed near the top and bottom, there is no criticism whatever to be made. The construction is exactly in line with the proposition I tried to bring out, namely, that when a continuous foundation is placed under a building it should act as a beam from one column to another, and the foundation must be of sufficient strength to allow this.

When the foundation is made entirely of concrete it should be made in the very best manner and of very great thickness. Not doing this has resulted in failures and results that were not satisfactory.

Prof. Wing alluded to the principle, used in Chicago, of separating

the foundation. I have here a drawing of the Old Colony Building, which is twenty stories high. The foundation plan shows the method of keeping the foundations entirely separate, even when the concrete bases extend close to each other. All the concrete areas have a distinct line of separation between them. This idea, when first started in Chicago, was used to such an extent that every single column had its own independent foundation, and with great care they kept them separate. Now they have commenced to group them more or less together.

The great Manhattan Building in New York, one of the highest of buildings, is constructed on a foundation in accordance with the ideas of Mr. Sooysmith, as represented in the paper he will present to the American Society of Civil Engineers, alluded to by Prof. Marx. His argument is on the line I have introduced here with regard to vibrations, only he had no reference whatever to earthquakes. It is that all buildings are subject to vibrations, however small, and sometimes they create considerable disturbance. Even the running of a hoisting donkey engine in the construction of a building causes vibrations that result in a settling of the building, The driving of piles will cause damage to adjoining buildings.

Prof. Wing.—I think my remarks in regard to building a continuous foundation have been misunderstood. Take a building like the Auditorium Building in Chicago, having a large tower in one portion of the building, and the other portions being of less weight and of less height, provided the foundation cannot cover a greater area than the building, if the building is constructed on a continuous foundation, there will be an inequality of pressure, and provision must be made for settlement where conditions like those met with in Chicago exist. In some cases the cantilever construction has to be used, as illustrated in Mr. Hunt's paper. If the building is simply of square construction, of equal weight in all portions, and of uniform height, I can see no objections to the plan of putting under a continuous foundation.

Mr. Hunt.—I am not an especial advocate of platform foundations, excepting when the conditions are favorable. I see no objections to this class of foundations in certain cases, and think they have distinct advantages. Of course, in a building of the size of the Mills Building in this city, it would be impractical to put it on a platform foundation covering the whole area. But the point I have tried to bring out in the paper is that the pressure upon the soil should be such that practically there would be no settlement. We know that all soils will support a certain amount of weight without compression, but they should not be overloaded. That is the principal point in all foundation practice; it is the underlying principle. It makes no difference if the foundation is upon soft mud, if we only establish the point of how much load it will carry without compression. The soil should not be overloaded.

Under certain conditions, where it would be too costly to secure more land, or something of that kind, of course that changes the situation. But if it is possible to avoid it, I think the soil should not be loaded so as to compress it to the point of perceptible settlement.

Mr. Leonard.—In regard to the settling of columns, a device has been used in Chicago, and I am told it is to be applied in New York, consisting of a space left for the hydraulic adjustment at the base of the column, so as to keep the building perfectly adjusted. As it tends to settle, the column is raised and steel wedges are inserted under it. It is taken care of in this way, and at the end of three or four years the building is supposed to have settled to a permanent position and to need no further adjustment.

Mr. Curtis.—On general principles there seems to be something about a foundation which is bound to settle if it is upon the natural soil, and I think we are always likely to have a higher respect for the scriptural man who founded upon a rock than for the man who built his house upon the unconfined sand.

To sum up the discussion, the idea suggested might be that the foundation that would meet all the objections to piles, or to timber and grillage, would be cylinders or wells sunk to the solid substratum and filled with concrete and capped with a concrete base for the whole structure, constructed somewhat upon Mr. Percy's plan, with steel near the bottom and near the top.

Mr. Percy.—I will say, Mr. President, that the nearest approach that I know of to such a method is a church in Paris, the Church of the Sacred Heart. They commenced its construction some years ago. It is a very large, heavy church, and situated on quite a high hill. They found they had a very unsatisfactory foundation. It was a mixture of clay with other materials, and was not at all satisfactory to the engineers, and they sank cylinders down 80 to 90 feet under all the main piers. They went down to the solid stratum and filled those cylinders with concrete, and instead of making a continuous platform under the entire building, heavy arches were sprung from one pier to another. These large cylinders were put down only at points where there would be great bearing. This is the nearest approach to what you suggest, which, I agree with you, would be the perfect foundation, answering all requirements that have yet been suggested.

LOCOMOTIVE COUNTERBALANCING.

By G. R. Henderson, Member of the Association of Engineers of Virginia.

[Read before the Association, June 27, 1896.*]

The subject of locomotive counterbalancing has recently been quite a favorite one, and there have been many valuable papers on this theme, but most, if not all of them, have been deficient in one particular; in that they have not clearly and simply indicated how to proceed with each part of the problem. For instance, one paper gave very carefully worked out formulæ for determining the effect of reciprocating weights, and how to correctly balance them, but the *proportion of reciprocating weights to balance* was passed by with a mere reference, as though of small consequence, when in reality it should be the fundamental question. In the following it is not the writer's intention to advance new theorems, but to select such points from previous papers (including those by Messrs. Parke and Sanderson before the New York and the Southern and Southwestern Railroad Clubs respectively) as, with a few logical suggestions, will place the subject in the hands of every Master Mechanic.

In developing these rules, three cardinal points have been borne in mind:

(1) The amount of reciprocating weight that can be left unbalanced may be a definite function of the total weight of the engine.

(2) The total pressure of wheel upon the rail must not exceed a certain definite amount depending upon the construction of bridges, weight of rail, etc.

(3) The vertical influence of the excess balance must never be sufficient to lift the wheel from the rail.

The first proposition is based on the assumption that the greater the mass, the greater may be the disturbing force without seriously affecting it, on account of its greater inertia.

The second is evidently a rational deduction, not needing any demonstration.

The third is necessary in order to avoid the wheels' jumping off the rail, thereby causing a real "hammer blow."

Starting with the above assumption, we arrive at the following conclusions:

A. Each wheel should be balanced for all revolving weights attached to it.

* Manuscript received July 20, 1896.—*Secretary, Ass'n of Eng. Socs.*

B. The connecting rod is to be considered as part revolving and part reciprocating weight; the proportion of weight of rod which is to be considered as revolving weight varies with the length of the rod as given below:

Length of rod in feet,	5	6	7 & 8	9 & 10	11 & 12
Proportion as revolving weight,	.57	.55	.53	.52	.51

C. The part of weight of connecting rod considered as revolving weight, should be entirely balanced in the main wheel.

D. The amount of reciprocating weight that can remain unbalanced without seriously affecting the locomotive may be found by the formula:

$$Wr = \frac{Wt}{360}$$

Wr = unbalanced reciprocating weight on one side (including portion of main rod).

Wt = weight of locomotive in working order.

E. The remainder of the reciprocating weights should be counterbalanced by dividing the amount equally between the driving wheels on the side, *provided* that the sum of the static weight on any one wheel, plus the centrifugal force of this overbalance, does not exceed the maximum pressure allowed for the particular type of engine in question at the maximum speed at which it will run. If some wheel loads are heavier than others, the lighter wheels may take a part of the overbalance which the heavier wheels cannot without exceeding the specified limit; nor must the centrifugal force exceed 75 per cent. of the static load on wheel.

F. The center of gravity of counterbalance must be opposite the crank.

G. The counterbalance should be brought out from the face of the wheel as far as clearance for the rods and proper design will permit.

H. The center of gravity of counterbalance should be placed as near the rim as possible, and the weight of the counterbalance reduced by this method.

I. Make reciprocating parts as light as possible.

Section *A* is self-evident. *B* is taken from one of the papers above referred to. *C* comes under the same ruling as Section *A*. In *D* the value $Wr = \frac{Wt}{360}$ is taken as representing good practice of the present day. It may be found that some different divisor will be more generally acceptable, but it is believed that the above will give good results.

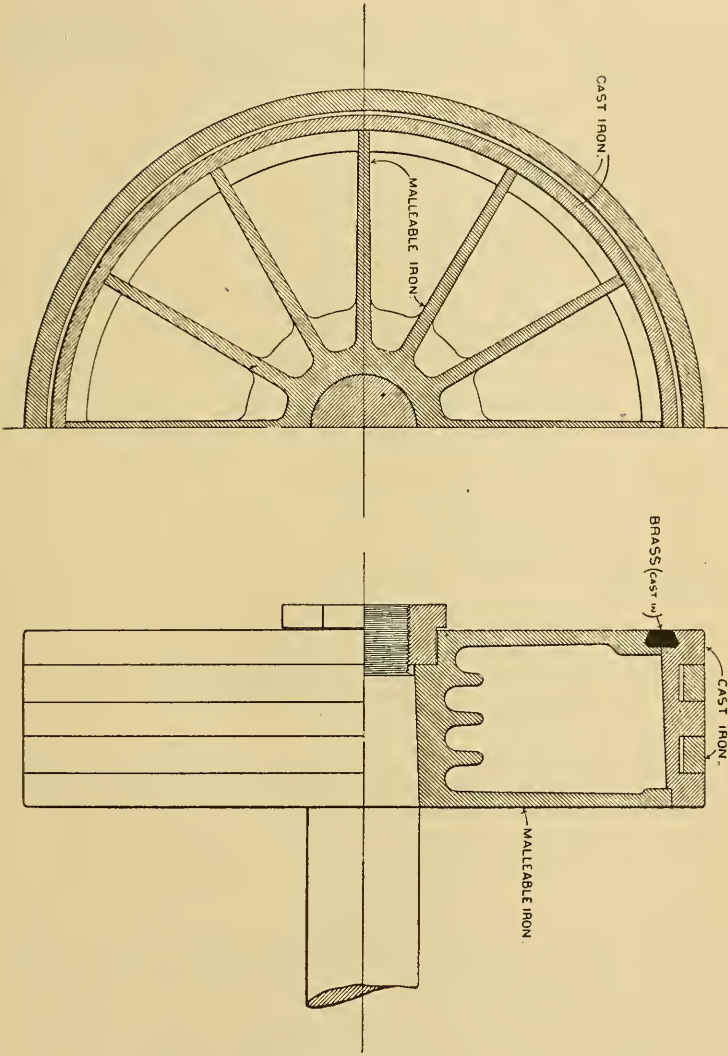

CAST IRON.
MALLEABLE IRON.
BRASS (CAST IN)
CAST IRON.
MALLEABLE IRON.

Fig. 1.

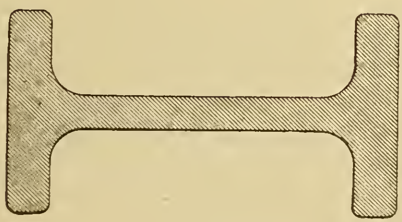


Fig. 2

To determine the centrifugal force for Section E, the following formula is obtained from Weisbach's "Mechanics of Engineering," Vol. I, page 609:

$$P = .00034\ u^2\ Gr.$$

where

$P =$ Centrifugal force.
$u =$ Revolutions per minute.
$G =$ Weight in pounds.
$r =$ Radius in feet.

Now letting

$S =$ Speed in miles per hour.
$D =$ Diameter of wheel in inches.

we have

$$u = \frac{S \times 5280 \times 12}{3.1416 \times D \times 60} = \frac{S \times 1056}{3.1416 \times D} = 336\,\frac{S}{D}$$

and

$$u^2 = 112896\,\frac{S^2}{D^2}$$

and substituting,

$$P = 38.4\frac{S^2}{D^2}\,Gr.$$

As in most locomotives $R = 1$, then we may put simply,

$$P = 38.4\,\frac{S^2}{D^2}\,G.$$

If now we assume that the maximum speed in miles per hour of the locomotive equals the diameter of driving wheel in inches, then,

$$\frac{S^2}{D^2} = 1 \text{ and } P = 38.4\ G, \text{ or say}$$
$$P = 40\ G.$$

It is also necessary to observe the limits of rail pressure. This will be different on various railroads, but on the Norfolk & Western it was taken as follows:

American type of locomotives . .	28,000 pounds per wheel.
Ten-wheel " "	26,000 " "
Consolidation " "	25,000 " "

[These loads are per wheel and not per axle or pair of wheels.]

Referring to Section F, it is found that the displacement of the counterbalance necessary to correct the effect of the weights and balance not being in the same vertical plane is so small on outside cylinder engines, that it is accurate enough to place the balance directly opposite the crank. By bringing the counterbalance out as suggested in G, it is possible to still more lessen the irregularity explained just above.

Sections H and I need no explanation.

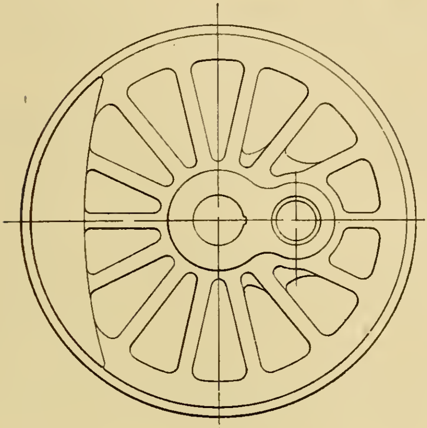


Fig 3

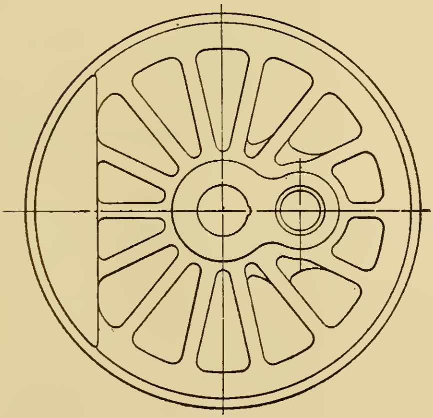


Fig 4.

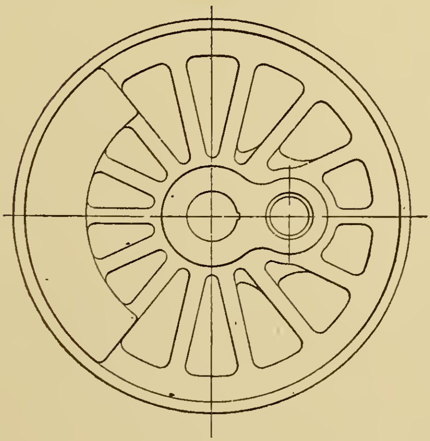


Fig 5

Having taken up these various points, the method of counterbalancing locomotives can now be reduced to the following:

RULE.

Divide total weight of engine by 360, this to be subtracted from reciprocating weights (including proportion of main rod) of one side of engine, and the remainder to be distributed among the driving wheels on one side.

The sum of forty times the amount of reciprocating weight allotted to any one wheel and the static load on the wheel, must not exceed the specified allowance for rail pressure, nor must forty times the amount of reciprocating weight balanced, exceed 75 per cent. of the static weight.

The weights to be put in each wheel will be inversely as the distance of center of gravity of counterbalance from center of wheel is to the crank radius, and must cover all revolving weights as well as the proper proportion of reciprocating weight.

In order to obtain the best results both for the engine and track, the following points should be remembered:

1. Keep the spread of cylinders as small as possible.
2. Make pistons of malleable iron, wrought iron or steel, to reduce weight.
3. Make piston rods of steel, and hollow.
4. Make crossheads of cast steel, of light ribbed construction.
5. Make the rods of steel and of an I section.
6. Keep counterbalances near the rim of wheel.
7. Keep counterbalance as far out as possible.

No. 1 can only be done when designing the engine.

No. 2 can be accomplished in various ways; however, the single-plate pistons have the objection that they freely transmit the heat of steam side to exhaust side of the piston, but double-plate pistons are not readily examined, as they should be, especially when very thin. Besides, a cast-iron wearing surface is desirable, while bolts and rivets are equally undesirable. A design of piston that promises very favorable results, and will meet all the above objections, is shown in Fig. 1. The center is malleable iron, and the wearing ring cast iron, the latter fitting against a shoulder at one side, while a brass retaining ring is cast in and opened out on the other side, making practically a single piece. It also takes ordinary cylinder heads.

For No. 3, the use of nickel-steel has been suggested.

No. 4 depends entirely on the arrangement of guides, etc.

For No. 5. Fig. 2 shows the favorite form.

No. 6 may be accomplished as shown in Figs. 3 and 4, in preference to Fig. 5.

No. 7 is limited by the clearance necessary for the rods, etc.

RIVETED JOINTS.

By Joseph R. Worcester, Member of the Boston Society of Civil Engineers.

[Read before the Society, April 15, 1896.*]

In spite of the fact that the tendency of the present time is more and more towards the use of riveted work in the construction of bridges and buildings, it is somewhat surprising that we hear of no change towards improvement in the customary methods of calculating the strength of riveted joints. Perhaps it will not be accepted without proof that the tendency of the times is in the direction just noted, but a little careful consideration will show this to be the fact, at least in this country.

The earliest iron bridges in general use hereabouts were constructed with stiff compression members made up of all sorts of rolled sections, or of cast-iron columns, tied together by means of forged rods, which, later on, were superseded by eyebars, and connected by means of pins. Likewise, until quite recently, the only iron used for framing buildings was in the shape of cast-iron columns upon which the beams were supported, the only connections being made by means of straps and bolts. As the science of bridge building developed, and the demands of railroads became more pressing, the speed of trains became greater and the loads heavier, it was found that the light pin-connected structures were gradually being rattled to pieces, the vibrations increasing to an alarming extent. In the endeavor to find some form of construction which would not show these defects, it was natural that we should turn our eyes towards the practice of European engineers and see what advantages we could gain by adopting more of the riveted form of construction, of which many fine examples were in use on the other side of the Atlantic.

It was at this period that a distinguished member of this Society, the late Edward S. Philbrick, designed the many plate girder bridges which have so well served their use in this vicinity for a generation, and which have never proved unequal to their original requirements until their metal was nearly eaten away by corrosion. One, if not more, of these stood until holes were rusted entirely through the web.

About this time some of our bridge companies began constructing riveted lattice bridges. These bridges were a distinct advance upon the earlier forms of pin-connected structures. Their trusses never became

* Manuscript received July 22, 1896.—*Secretary, Ass'n of Eng. Socs.*

shaky in spite of the unscientific connections, the frightfully bad intersections of web members, and the faulty and incomplete systems of bracing; but whenever they have given out, it has been on account of the fact that the floor beams and stringers were not of sufficient strength to carry the increased loads, or the connections in the floor system were not as efficient as the trusses. Many examples of these bridges are doing good service to-day, though probably most of them have had their floor system strengthened or wholly renewed.

Within a short time one of our members has told us that comparing two bridges, a pin and a riveted, of equal theoretical strength, the riveted bridge was very much the stiffer, and consequently, in his opinion, the better bridge.

As time went on, and experience accumulated, we find railroads specifying that riveted girders and lattice trusses shall be required for longer and longer spans, until now we see riveted joints containing two hundred and fifty rivets, to be driven in the field, used in the center truss of a heavy four-track drawbridge, and we see our railroads using plate girders for highway bridges of one hundred feet span. We see all modern specifications for railroad bridges requiring riveted lateral bracing at the track level; while, upon the other side of the water, we see such a bridge as the Firth of Forth riveted throughout, in spite of the fact that our eyebar practice was well understood and carefully considered in England at the time this structure was built.

In our building construction we see the same tendency. The loose strap and bolt attachments are giving place to riveted connections, and the cast iron columns are being superseded by rolled steel sections which will permit better riveted connections.

The object which we are striving for and gaining by these changes is the *rigidity* which seems to be inherent in riveted work.

Notwithstanding this tendency towards the increased use of riveted work, we are still using the same methods of proportioning riveted joints that have been in common practice for fifteen years, in spite of the fact that this practice involves many manifest absurdities.

The earliest authorities on the use of rivets, with more reason than we are in the habit of accrediting to them, took into account only the shearing value of the rivet, but for a long time now we have been taught to use either the shearing strength or the bearing value of the rivet against the metal opposing it, whenever the latter appears to be less than the former. These two strains are all that are considered in modern practice, though some have even gone so far as to consider the fiber strain caused by the bending moment.

In proper riveted work the writer ventures the assertion that neither one nor the other of these strains is exerted to any extent, and

it is with the intention of proving this assertion that the present paper is presented. It is not denied that before riveted joints will fail both bearing and shear will come into play, but when we are considering *proper* work we mean a class which is not on the point of failure and has not even reached the limit of elasticity, which it does as soon as a joint shows any permanent set. The rigidity, which is the essential characteristic of this form of construction, would not appear if rivets allowed a motion to take place between the thicknesses of metal connected. The force, therefore, which should be considered in designing the riveted joints, is that force which is exerted by the rivets to restrain the parts from all motion and to hold them in the precise position in which they are riveted.

The easiest way to demonstrate that it is neither bearing nor shearing strength which causes the rigidity of riveted work, is theoretically, though we can quote also a number of practical illustrations which help to confirm this position. When a rivet is driven hot it is supposed to fill the hole, which is usually one-sixteenth to one-eighth inch larger in diameter than the rivet. This filling of the hole which specifications always stipulate, is more or less perfectly accomplished. When the holes are fairly concentric in the various thicknesses through which the rivet passes, the punch and die being not far from the same size, and the rivets are driven by a powerful machine, the metal will upset into the hole for a considerable distance until it apparently fills the void spaces; but when the rivets are driven by hand or when the holes have large tapers, or when the rivet passes through a number of thicknesses, there are sure to be voids of considerable dimensions. Under the most favorable conditions, however, we must remember that at the time the rivet is driven it is at a much higher temperature than the surrounding metal, and as it cools it must inevitably contract, and in so doing draw away from the surface of the hole against which it was pressed. A section cut through a riveted joint in the axis of the rivet will show in a superficial examination the rivet to be in close contact with the surrounding metal, but with a magnifying-glass, or with the point of a needle, it is very easy to see that the contact between the rivet and the surrounding metal is not as close as that between the head of the rivet and the surface of the plate, or between the various thicknesses of plate tied together.

It is this fact of there being a little play around the rivets which prevents their acting either by bearing or shear until there has been a little slip between the plates, that is, until the joint has passed its limit of elasticity. Up to the point when the slip occurs the force which prevents motion must be the friction caused by the pinching together of the surfaces by the rivet heads. That this frictional resistance must

be the force upon which we depend for the rigidity of our riveted structures may be practically shown in various ways. In the first place a great many experiments have been made on testing machines which invariably show that as the strain is applied the joint in the first place stretches only just as much as the metal itself stretches and in direct proportion to the amount of the strain. During this period, if the tension is relaxed the specimen returns to its unstrained position, but as soon as the friction is overcome a certain motion occurs, the

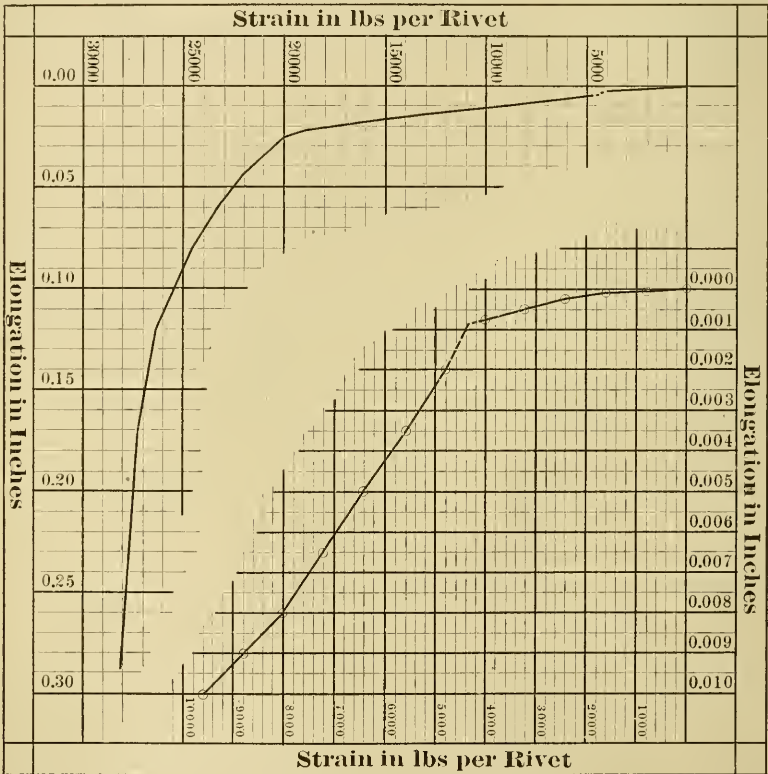


Diagram Showing Slip of Rivets.

extent of which depends upon the void spaces around the rivet. After this the specimen shows a permanent set and the joint will go on stretching in an irregular fashion, as the metal around the rivet comes into full bearing. The diagram herewith presented illustrates this slipping very forcibly. It is taken from a series of experiments tried at the Watertown Arsenal in 1886, and is a fair sample of the several hundred tests made at that time.

In this diagram, the curve on the left shows the total elongation of the joint until nearly the time when failure takes place, the ordinates indicating the amount of strain per rivet, and the abscissæ showing the stretch of the joint. The curve on the right shows the early part of the same curve on a magnified scale, the points at which the elongation is recorded being indicated by circles.

As will be observed, up to a strain of about 4,000 pounds per rivet, the elongation is approximately proportioned to the strain, but at about 4,300 pounds a sudden slip occurs. The exact point of slip is not recorded, the curve at that point being indicated by broken lines, which are produced by extending the general direction of the lines above and below the nearest observations.

Another method of showing practically that rivets do really hold by means of friction is by considering what would occur in the case of a plate girder with a large number of flange plates. In such a place as this it is impossible for any ordinary riveting machine to more than partially fill the holes. Any one who has had experience in cutting out very long rivets must have noted that they are easier to back out than short rivets through two or three thicknesses. The reason is that when the pressure of the riveter is applied to the end of the rivet it begins to upset at the extreme point, and as it upsets it fills first the part of the hole nearest the driving-head. As this fills out into the irregularities of the whole, it jams, so that it is impossible to force enough metal in through the hole to fill out the voids near the head end. We have, therefore, in this case even more play around the rivets than would occur where the hot rivet was completely upset. But these rivets have to transfer the strain from the flange angles to the outside plate, often a distance of several inches. If we imagine such a plate girder put together with loose-fitting pins, which indeed it would be were it not for the friction, it is evident that the girder would have to get a very considerable set, and the web and angles perhaps fail before the whole flange would come into play, if it could at all.

Another example of about the same action is where we see connections made through loose fillers. We can often find in good practice the end-uprights of a stringer which transfer the whole shear from the stringer web to the supporting floor beam, packed out to the thickness of the flange angle by means of a bar of the same width as the upright. In this case if there were not friction exerted it is evident that the rivets would bend quite appreciably and thus allow the stringer to drop before bearing and shear came into play. Another evidence of the fact that it is the friction which is effective may be found in numerous examples of girders which were formerly constructed with very thin webs, and which with a bearing strain of the rivet against the web up to and

above the elastic limit of the material, have not shown the least sign of motion. The writer has in mind the case of a bridge on the Old Colony Railroad which was removed a few years ago, where this bearing strain caused by the every-day traffic of the railroad, amounted to 20,000 to 30,000 pounds per square inch, without allowing anything for the effect of impact.

That this fact of frictional resistance has been really recognized by engineers can be evidenced by the sensible though not very common practice of allowing a greater strain in bearing on metal enclosed by thicknesses acting in the opposite direction, than where not so enclosed. This practice appears to have originated in the thought that in a case of single shear the rivet would naturally bear upon the edge of the hole nearest the plane of shear with greater force than upon the other side of the plate, but the effect of the specification is in the line of taking account of the friction, of which we naturally have twice as much in the case of enclosed bearing as in that of not enclosed, and this fact is often advanced as an excuse for the practice.

The most conclusive experiments on the question of friction in riveted joints that the writer is aware of, were made in France two years ago, by M. Dupuy, Inspector-General of Bridges and Highways, who was intrusted by the Ministry of Public Works with the duty of making special inquiry into the causes of deterioration of metallic structures. A full account of his experiments and conclusions was presented in *Les Annales des Ponts et Chaussées*, January, 1895.

The experiments were conducted with the greatest care, especially to determine the strength of the rivet before reaching the limit of elasticity. The conclusions which he reached are based upon a very clear and convincing argument drawn from the experiments. The following points which he arrived at seem to be unassailable:

1. Rivets are stretched bars undergoing a tensile strain higher than their initial limit of elasticity. The fibers of the circumference appear to be more stretched than those of the center.

2. Rivets do not quite fill the holes, but exercise a very strong clamping effect which causes between the plates a resistance to slipping equivalent to a welding.

3. The resistance to slipping of riveted plates increases as the limit of elasticity of the metal of which the rivets are composed increases.

4. The limit of resistance to slipping is extremely variable. The causes of this variation appearing very numerous and depending (*a*) upon the nature of the metal of which the rivets are composed, and (*b*) upon the temperature at which the rivets are driven; (*c*) upon the temperature at the completion of the operation of driving; (*d*) the method of riveting; (*e*) the manner in which the operation is conducted.

5. The resistances to slipping upon which we can count in practice in connections composed of three rivets or more, in pounds per square inch of rivet section to be sheared, are shown by the following table:

	Steel Rivets.		Iron Rivets.	
Original Limit of Elasticity in pounds per square inch.	29,900	32,700	25,600	29,900
Hand-Driven Rivets heated to a bright red heat, the dies leaving no mark on the plates, the operation being finished when the driven head has become black.	6,400	7,110	5,690	6,680
Power Rivets heated to a white heat, driven with a pressure equal to 85,000 pounds per square inch of rivet section, the pressure being maintained until the head has become black.	8,530	9,390	7,110	8,250

The limits of elasticity mentioned above are those which are found in testing specimens previously heated to a dull red.

The metals employed in making rivets should have an elongation of at least 12 per cent. for iron and 18 per cent. for steel.

6. If a riveted joint is subjected to a strain sufficient to cause the thicknesses to slide, even if the motion is enough to distort the rivets, the distortion will remain after the strain is removed, but it appears that no further distortion can be produced without a greater strain than that which caused the original distortion.

After establishing his conclusions in regard to riveting, M. Dupuy goes on to deduce rules for designing bridges, which, for the most part, are admirable, especially those relating to the desirability of avoiding secondary strains as far as possible by making perfect intersections of diagonals with chords, etc., but he draws some conclusions which seem to run counter to the ideas which have been gaining ground in American practice, and which require further demonstration before we can accept them. For instance, he recommends that panels shall not be made longer than 11′ 6″ to 13′ 0″. He also recommends that in double Warren systems of trussing, verticals shall be used at each panel point to connect the two systems and so equalize the strains. He further advises that bridges of several spans shall be made continuous over the piers where there is no danger of a settlement, though he would not ad-

vise the continuity to extend over so great a number of spans as to induce large strains on account of temperature changes.

The writer has attempted, by careful examination of the reports of tests at the Watertown Arsenal, to verify the experiments of M. Dupuy, but the examination has not proved altogether satisfactory, for the reason that the reports do not indicate precisely the point at which the first slip takes place. The stretch of the specimen is indicated at certain intervals, and it is easy to see that at some point the slipping takes place, but the points of observation are not sufficiently close together to indicate the exact point where the motion begins to occur.

Another reason why the experiments are not wholly satisfactory is that the bulk of the experiments have been tried upon a single row of rivets driven as in boiler shells, the rivets in most cases being much closer together than three diameters center to center, under which circumstances the frictional action does not seem to work to so great an advantage.

The following tables give the average of a large number of tests, the strain indicated being the shear per rivet at the last observation point before the slip occurs :

TABLE I.

Earlier Tests of Rivets in Single Shear.

Force required to produce a slip in pounds.

Diameter of Rivet.		Thickness of Metal.			
		$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$
Iron Rivets . . .	$\frac{7}{16}$	1775			
	$\frac{5}{8}$	3810			
	$\frac{11}{16}$		3904		
	$\frac{3}{4}$			5200	
	$\frac{15}{16}$			7000	
	1				8625
Steel Rivets . . .	$\frac{7}{16}$	3750			
	$\frac{5}{8}$	4000			
	$\frac{11}{16}$		4333		
	$\frac{3}{4}$		5000		

TABLE II.

Later Tests of Iron Rivets in Double Shear.

Force required to cause a slip in pounds.

Diameter of Rivet.	Thickness of Metal.				
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$
$\frac{9}{16}$	4012	4150			
$\frac{11}{16}$	4000	5012	4525		
$\frac{13}{16}$	3833	5250	6130	6300	
$\frac{15}{16}$	4400	4740	5400	7200	6700

It appears from these tests that, while the thickness of the metal against which the rivet bears plays little part in the frictional strength of the joint, it seems as if it is necessary to get a tolerable length of rivet before the clamping effect can be fully developed, for we see in no case an increase of strength at all proportional to the thickness. We even find for the thickest plates less strength in some cases than with thinner metal. It does, however, appear that with the thinnest metal, that is, where the rivet is very short, we fail to get quite so high results as with a little longer rivets.

The explanation of the fact that, in these tests, the larger diameters of rivet do not seem to be as efficacious as the smaller, may be partly due to the fact that the pitch of rivets is more suitable for the small than the large sizes, and they should not be considered as discrediting M. Dupuy's results on account of this disagreement, but it is exceedingly desirable that further experiments may be tried, to enable us to determine whether he is correct in assuming that the frictional strength is proportional to the area of the rivet section.

It is interesting to note that the behavior of the specimens under test was exactly the same as the French experiments showed. This is clearly described in the following extract from the report of Mr. J. E. Howard, C. E., which accompanies the tests quoted above:

"The progress of the test of a joint is generally marked by three well-defined periods. In the first period greatest rigidity is found, and it is thought that the joint is now held entirely by the friction of the rivet heads, and the movement of the joint is principally that due to the elasticity of the metal.

"The second period is distinguished by a rapid increase of stretch of the joint, attributed to the overcoming of the friction under the rivet heads and closing up any clearance about the rivets, bringing them into

bearing condition against the fronts of the rivet holes. Rivets, which are said to fill the holes, can hardly do so completely, on account of the contraction of the metal of the rivet from a higher temperature than that of the plate, after the rivet is driven.

" After a brief interval, the movement of the joint is retarded, and the third period is reached. The stretch of the joint is now believed to be due to the distortion of the rivet holes and the rivets themselves.

"The movement begins slowly, and so continues till the elastic limit of the metal about the rivet holes is passed, and general flow takes place over the entire cross-section, and rupture is reached."

If we, then, assume that the experiments, both here and in France, are sufficient to establish the fact that the first action of the rivets is to hold by friction, the question arises, whether it is not possible to adopt specifications based upon this force, which will give more perfect results than those attained by using bearing and shear.

A few of the objections are:

1. Want of tightness of the rivet. Of course, if the rivet is not tight it does not hold by friction, but this is no argument against using friction in proportioning the joints, because a loose rivet is bad anyway, and one such in an otherwise perfect joint cannot be said to do any good, whether the rivets are figured by bearing or shear.

2. The rivet may be driven through thicknesses which are not in perfect contact, and from the stiffness of the plate may appear to be tight when it is not really exerting as much pressure as the metal is capable of. This defect, while it certainly may be a serious one, especially when hand-driven, could not exist if the thicknesses were properly clamped together during the process of driving, and it can only be said that it is as much a source of weakness, no matter how the joint was originally proportioned.

3. Another objection which occurs to the writer, is the doubt whether a joint may be lubricated by oil or paint applied to the contact-surface to such an extent as to appreciably alter the coefficient of friction. As to this point, it is exceedingly desirable that more experiments may be tried, and possibly some of the members present can supply information on the point which the writer has not succeeded in finding. Without mentioning the fact that it is of very doubtful utility to apply paint or oil to surfaces which are to be riveted in close contact, it is the firm conviction of the writer that the heat of the rivet and the squeezing effect produced by the process of riveting entirely dissipate any effects from this cause.

Among the many advantages may be mentioned the following:

1. Simplicity. Evidently it would be much simpler if we could neglect the thickness of plates and consider only one value for a single shear of a certain-sized rivet.

2. Comparative accuracy. While we may not be able to determine exactly the frictional strength of a rivet as long as it actually holds by friction, it does not seem as if we could gain much by considering two other functions, such as bearing and shear, which do not really act at all.

3. The doubtful and uncertain questions arising in case of indirect transmission would be entirely avoided.

4. When we realize that rivets hold by friction we free our minds at once from two complications which are apt to be troublesome in designing, viz., the questions of fatigue and reversal of strains, for neither of these has any effect upon a joint holding by friction.

5. A true conception of how rivets hold would prevent some details of construction which are now quite frequent. It is often convenient to drive two knees (between which a gusset or a beam web is to be inserted later) to some supporting members, the knees being purposely left a little wide apart to allow easy assembling. It is sometimes found convenient to drive the flange rivets of a girder before the web rivets, and the writer has even known of the flange angles of a large girder being shop-riveted to the flange plates and shipped separate from the web plates, which were afterward inserted and hand-riveted. Such practices which destroy all possible frictional action, could not be tolerated if we counted on this force.

6. It may be well to note that in adopting this method of figuring we are running no risks, because in the few cases in which this method would allow a larger strain on the rivet than the old method, even though the friction should give out and the rivet should slip so that the bearing would come into play, we have abundant evidence that no disaster can result worse than a slight set in the joint.

As to the safe values to allow for the frictional resistance, it may simplify the consideration to think of the force as depending upon the clamping power of the rivet multiplied by the coefficient of friction.

The clamping effect is produced by the contraction of the rivet in cooling. If the pressure of the machine, or of the bolts in the case of hand-riveting, is sufficient to bring the thicknesses riveted into close contact and the rivet is properly heated, the contraction is greater than the possible elongation of the metal of the rivet within the elastic limit. The result is that the strain in the rivet is just enough to stretch it. M. Dupuy verified this fact by some very ingenious experiments, which are fully described in his paper above quoted. This being the case, we have one of the elements upon which the friction depends, determined with considerable precision. The other element, the coefficient, must necessarily be determined experimentally. Judging by experiments of Rennie, it would be in the neighborhood of $\frac{4}{10}$ and considering the fact

that the pressure is very great, and that there is always a certain amount of unevenness around a hole which would tend to increase the coefficient, it seems as if we could count on about this figure with reasonable certainty. This is confirmed by the result of Watertown tests of rivets driven in slotted holes. In this case it was found that $\frac{5}{8}$ rivets in single shear required a force of about 5,000 pounds to produce a slip. This is equivalent to a strain of 15,340 pounds per square inch, or probably a coefficient of friction of $\frac{6}{10}$. The elastic limit multiplied by $\frac{4}{10}$ would give for steel about 12,000 pounds per square inch of rivet section, and for iron about 10,000 pounds, but these values are above the usual allow-

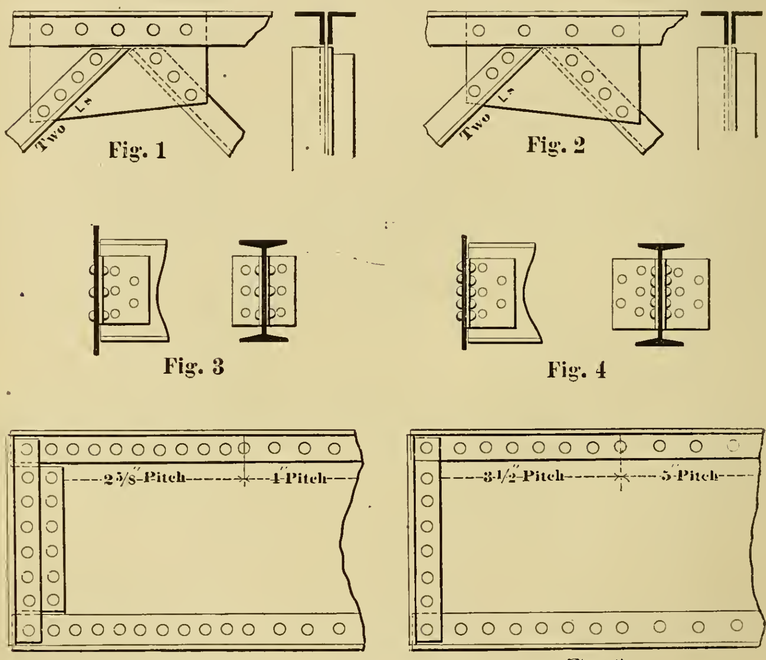


Fig. 1 Fig. 2 Fig. 3 Fig. 4 Fig. 5 Fig. 6

Sketches of Details Designed with and without Regard to Friction.

ances for shear. It seems, therefore, to the writer, that if we should only figure out rivets for shear, recognizing all the while that it is the friction which is holding and assigning safe shearing units, we should be approaching much nearer a truly economical and consistent design than by considering bearing.

This may be a step backward, but it is possible that in departing from the rules of our predecessors we have not gained as much as we think.

While it is not the object of this paper to go into the question of

a proper specification, it may be well to state that considering the question of friction, there ought to be more distinction made than is usually done between allowable shear on hand-driven and machine-driven rivets.

A few sketches are herewith presented showing ordinary bridge connections as they would appear designed in the common way and also by means of frictional resistance.

The sketches on the left, Figs. 1, 3 and 5, represent ordinary joints calculated for bearing and shear as in ordinary specifications. Those on the right, Figs. 2, 4 and 6, show the same joints calculated upon the basis of frictional resistance.

As will be noticed, the number of rivets required is sometimes more and sometimes less, when the friction is taken into account. While on the average the number of rivets used may be about the same, it is thought that the distribution can be improved and more strength gained by considering the friction.

DISCUSSION.

By Edward S. Shaw.—The subject of the increase of strength of riveted joints under ordinary working strains, due to the friction of the contact-surfaces of the plates or shapes riveted together, and the question of how much of the strength of the joint is due to this friction, are interesting matters, which doubtless have not heretofore had the consideration that they deserve from engineering writers and experimenters.

The existence of this frictional resistance has long been known, but the older authors of engineering text-books, so far as the knowledge and memory of the writer goes, dismissed the subject in much the same way as Trautwine, who says: "The friction between the plates in a lap, or between the plates and the covers in a butt, produced by their being pressed tightly together by the contraction of the rivets in cooling, adds much to the strength of a joint while new, perhaps as much as 1.5 to 3 tons per square inch of circular section of all the rivets in a lap, or of all on one side of a single-cover butt; or 3 to 6 tons of all on one side of a double-cover butt. In quiet structures, this friction might continue to exist, either wholly or in part, for an indefinite period; but in bridges, etc., subject to violent and incessant jarring and tremor, it is probably soon diminished or entirely dissipated. Hence good authorities recommend not to rely on it, and it is therefore omitted in what follows."

If we admit the universal existence of this friction as a factor, even

if not the controlling one, in determining the number and strength of the rivets, it would seem also to have an appreciable effect upon the net strength of the plates connected by splices in a riveted bridge joint, for beyond the last rivet or last line of rivets in the joint, the splice plates extend, pressed to the main plates with a certain strength of grip which must be overcome before the plate can tear on the outer rivet holes.

Mr. Worcester should have the thanks of all engineers interested in bridge designing and construction, for the careful and able manner in which he has collected the somewhat meager data existing upon this subject and drawn general conclusions therefrom.

It is to be hoped that he will continue his investigations so far as to present a complete and rational method, at least for the spacing of rivets in the webs and flanges of plate girder bridges.

In regard to the general application of this method, there would seem to be more than a few difficulties, especially in connection with field-riveting.

We have learned to regard the proper upsetting of the rivet, and filling the hole as well as possible, as of utmost importance. It is, however, too much to expect of the average field-riveting gang, that they will get perfect heads on all their rivets, and make them tight and well upset in the holes at the same time ; and it has been the practice of the writer to accept a limited number of unsatisfactory rivet heads, especially in difficult positions for driving, provided that the rivets were otherwise well driven.

The substitution of the author's method would require a reversal of this rule, for if the plates are to be held together by the grip of the rivet heads, it is essential that there should be no small, flat or thin-edged or badly eccentric rivet heads, for though at first the grip may be sufficient with a flat head or small bearing surface, yet rust and other causes are very liable to diminish or destroy this surface and grip.

The writer must express views in opposition to the fourth conclusion of the author, viz.: " When we realize that rivets hold by friction, we free our minds at once from two complications which are apt to be troublesome in designing, viz.: the question of fatigue and reversal of strains, for neither of these has any effect upon a joint holding by friction."

In contradiction to this may be placed the old theory, well expressed by the statement of Trautwine quoted above in regard to the effects of impact and vibration ("jarring and tremor"), the same causes, or results of the same causes, which are supposed to produce fatigue in the main members and which are operative in necessitating a diminution of the allowable strains, with increase of live load or reversal of strain. The rivets being in a high state of tension, (according to the French experimenter quoted by the author, beyond the primary elastic

limit of the rivet material) it would seem to the writer that they must be especially sensitive to the fatiguing effects of dynamic influences acting upon them through the media of the plates or parts connected, while the action upon the rivets caused by the movements of extension or compression of the members connected, however slight, will apparently, in time, tend to diminish their grip and the resulting friction, in about the same ratio that the microscopic molecular changes of deterioration or fatigue in the main members are produced.

Therefore, if it is proper to vary the unit strains in the principal members with variations in the ratio of minimum to maximum, then it would seem to be equally proper and thoroughly consistent to vary them in a proportional or similar manner in the rivets.

By John C. Moses.—The so-called "factor of safety" has frequently been termed a "factor of ignorance," and the writer of the paper has clearly shown the truthfulness of the charge in the case under discussion. But no engineer can really be satisfied with factors of ignorance, and so we find him constantly trying to eliminate the guess work and substitute what is known to be correct. The most satisfactory method is to make experiments and reason from them as a basis. It is positively appalling to think of the amount of mental labor involved in arriving at our present theories of the action of structures—mental labor that a few actual experiments would so greatly assist. These experiments are not made because, on the one hand, no manufacturer is sufficiently disinterested to do a work that will not bring him a direct gain unshared by others; and, on the other hand, no purchaser feels called upon to pay for experiments that will benefit everybody as well as himself. No one questions the need, but they say it benefits everyone, and so everyone should help pay for it. Consequently it is not done. The writer speaks of the possible effects of paint as a lubricant of riveted joints—he *thinks* its effect is inappreciable, but perhaps ten years from now someone will spend five hundred dollars and find out. We have waited many years for the experiments on riveted joints in tension cited in the paper. The French experiments were made possible by government aid, but we cannot expect that in this country.

A recent article in the *Engineering News* demonstrated that the standard beam connections in common use are only worth from one-fourth to one-half the commonly assumed values "if the effect of friction be neglected." We all know this, but we also know that rivets have a value aside from their shearing and bearing values, and we *guess* that it is enough to counteract the effects of eccentricity. This is a very crude approximation. A hundred beams connected as in actual practice, and loaded until the elastic limits of the joints were reached, would give us a new basis for our theories, and probably enable us to save a pound or two

of metal in a joint, while at the same time making it a stronger piece of work. To accomplish a better result with less money is to be an engineer in the truest sense. How long would it take to pay for the experiments if an average of one pound a joint could be saved as the result? Meantime the profession is daily making a ridiculous assumption, because no one makes the experiments. What better work could be found for an engineering society to do? Manufacturers can be interested if some one will take the initiative. Testing machines and men to run them can be found in Cambridge and Boston, and the cost of the materials need not be great. Our last president has suggested dividing our Society into groups, each one in turn furnishing the paper of an evening. The report of the Committee on Standard Connections for Beams would make an interesting subject for some of us, and if properly done it would be a distinct addition to engineering knowledge and a new demonstration of the usefulness of this Society. There are, of course, many other questions awaiting similar solutions; this one is merely suggested as an example, the solution of which is indicated in the evening's paper. One cannot help thinking, as he reads that paper, how much more satisfactory it would be if the author had been able to base it on more extensive experimental data. If he could have positively told us that it was correct to use the shearing values of rivets when spacing them in flanges of girders, the information would save several per cent. of the cost of the work done in the establishment with which the writer is connected. As it is we will feel safe to use a somewhat higher bearing value for rivets than has been our custom heretofore.

By James E. Howard. — That riveted plates are held together by a very substantial gripping pressure exerted by the rivets in cooling, there can be little doubt. A consideration of the coefficient of expansion by heat shows that rivets need cool only over a limited range of temperature in order to be strained to their elastic limit. This zone of temperature is of such limited range that the known changes in the modulus of elasticity and elastic limit under higher temperatures has but slight influence in the results, and it is believed that under favorable conditions rivets in their final state may be left gripping the plates of a joint with a force nearly or quite coincident with their elastic limits.

In hydraulic riveting the conditions seem favorable for reaching good results. Rivets in their upset state may, however, have a lower elastic limit in consequence of the upsetting than possessed by the metal in a rolled bar.

That rivets are strained nearly to their elastic limit the tests of the joints seem to afford some proof.

In the early stages of a test it is frequently observed that the scale starts off the rivet heads, and when this occurs under a comparatively

low stress on the joint it is taken to signify that the rivets were nearly ready to scale when the joint was at rest.

A critical comparison of the strains developed in a joint and those which should be developed in the solid plate, adopting a modulus of elasticity of 30,000,000 pounds per square inch, shows that joints very frequently elongate more than a solid plate should elongate, and that permanent sets appear early in the joint.

So far as these minute changes in shape are at present explainable it would appear that while in the main frictional resistance prevents general slipping of the plates, yet slight distortions are permitted to occur.

It is difficult to ascribe an adequate cause for this behavior. Perhaps it signifies the release of internal strains, where conflicting strains existed due to the contraction of the parts while cooling after riveting.

Possibly the most advantageous case for maximum frictional resistance is found in a joint containing a single row of rivets, running at right angles to the line of applied stresses.

Where several rows of rivets are used, the outer rows necessarily are obliged to permit some movement of the joint in order to bring the inside rows into action. Just why Mr. Worcester considers experiments made in a single row of rivets unsatisfactory (see page 40) does not appear to be explained, nor why the spacing should exceed three diameters for the frictional resistance to work to advantage. Data seems needed to show what part of the maximum frictional resistance existing or supposed to exist in a joint is available for use in a structure. Perhaps an experimental inquiry into the behavior of joints exposed to alternate and variable stresses and also vibratory influences would aid in answering this question.

By Gaetano Lanza.—(1) The chief objection to Mr. Worcester's theory is, of course, the fact which Mr. Howard has already mentioned, that the stretch due to the application of the load on a riveted joint is always greater than that due to the stretch of the metal, and hence that there is no load which does not cause slipping.

(2) Moreover, if, as I suppose Mr. Worcester must have tried to do, we seek for a load at which the rate of slipping increases very decidedly, we shall find, in many cases, that the increase in the rate of slipping is gradual.

(3) Were the action such as Mr. Worcester describes, it would be a dangerous proceeding to design joints on the basis of the friction alone, without considering their strength, as it would be possible, on that theory, to make a joint having more frictional resistance than strength, or, at least, where the frictional resistance formed a large proportion of the strength and the factor of safety was very small.

(4) The experiments of Dupuy seem to me to be too few in number, and also, for the most part, to have been made on joints with too few rivets to warrant the conclusions drawn. The determinations of stretch were much coarser than those of the tests at Watertown arsenal, and the fourth group of joints were tested on a machine in which piston friction was allowed to vitiate the results.

Of the other three groups, the first was the only one where there were as many as four rivets on each side of the joint, and these were hand-riveted; the number of tests being four. In the second group of seven joints there was only one rivet on each side of the joint, and in the third group of twelve joints there were only two rivets on each side of the joint.

By J. P. Snow.—It seems to me that the web rivets in plate girder flanges get the advantage of frictional resistance more than those in almost any other situation. If the web tends to fail by bearing, that is if it starts to buckle around the rivet hole, the angles held by the flange plates prevent it and this action tends to increase the friction rather than to diminish it, as would be the case in a simple connection where the rivets act in single shear. Oftentimes, when designing under the usual specifications, the thickness of webs is governed by the rivet bearing near the ends. I think that the usual unit strains might be safely exceeded in these cases. So far as I have been able to judge from observing old girders, the thickness of webs is a part that may be allowed to vary more widely from established units than other members of the structure, and it is hardly conceivable that one could be so badly designed as to fail from the insufficiency of flange rivets. It must be the friction that helps in these cases, and I believe that it is legitimate to take advantage of what we know must exist in proportioning new work, although I acknowledge I have never had the courage to raise these units appreciably.

It is troublesome to arrange a general specification so that it will cover extreme cases without running into absurd results; this makes it difficult to provide for all the varying cases where it seems advisable to depend wholly or partly on the friction. The cautionary legend that Mr. Cooper puts at the head of his Standard Specification is well placed; much depends on the judgment of the designer. It is, however, feasible to provide for a different unit for hand-driven and machine-driven rivets and for rivets in single and enclosed bearing. The practice on the Boston and Maine Railroad is to allow 25 per cent. more on machine-driven than on hand-driven rivets and 25 per cent. more on rivets enclosed between two thicknesses than on those acting in single shear.

I heartily agree with the paper in the matter of making these allowances, and in crediting the friction with a generous ratio of its

ultimate value in the case of web rivets in plate girders. In cases of connections where the rivets act in single shear, however, especially when they are hand-driven, it will hardly do to allow more on a rivet than the bearing area or shear could properly carry, because these surely are the ultimate dependence if the rivets from any cause become loose. The examples shown by the author call for more rivets in this class of connections than the usual rule. How this would work out for different thicknesses cannot be told without a definite specification. It is probable that in thin metal less rivets would be called for by the new rule than the old. The usual number should be reduced with caution, for although it may be that rivets do not get loose much worse in thin metal than in thick, yet after a rivet is loose the thin metal goes to destruction much the faster, and it is hardly to be expected that any set of rules will prevent rivets getting loose occasionally. This is off the question somewhat perhaps because the paper considers "proper rivets only" and those which become loose cannot be called proper; but we must deal with actual conditions and try to design work to meet them.

In the structures under consideration we must unfortunately depend on hand-driven rivets and generally on those acting in single shear to perform the most important function of all, that is, to connect the various members together; while the rivets of less importance, that is, those holding together the integral parts of a member, can be machine-driven. This condition operates in several ways to the disadvantage of riveted trusses. Being so vitally important, and at the same time of so low efficiency, we must have plenty of them. They are what hold our structures together and we must not depend too much on so uncertain an element as friction.

As to the superior rigidity of riveted trusses alluded to in the paper, it is probable that the form of section has as much to do with it as the style of connection. The flanged sections in pin trusses vibrate but little, while flat bars in riveted ones are but little more rigid than if connected by pins. I think if satisfactory joints could be arranged in pin-connected trusses when using flanged sections like angles, zees and channels throughout, the resulting structure would be as satisfactory in the matter of rigidity as it would be if riveted.

It is very interesting to compare European practice with ours, as is done by the author. On the whole it seems to me that the Americans have been the most ready to drop what is bad in their past practice, and to adopt the good from foreign design. In our solid-floor bridges, we have certainly improved on the English designs, and in our long-panel trusses we have dropped most of the objectionable features of early American flimsiness. The Europeans seem to cling to their clumsy short panels. Some American designers claim that long panels only are right, that

panels less than 20 feet should be frowned upon. It does not seem to me that the length of panel *per se* affects the efficiency of the bridge at all. This element as well as the style of the bridge should be adjusted to meet the conditions. Within twelve months we have designed bridges for the Boston and Maine Railroad with panels varying from 2 feet to 23 feet, and so far as I know they are equally strong and serviceable. It was surely our endeavor to make them exactly equivalent. Our practice is equally varied in the matter of pin and riveted work. I believe that each has its place. He who will use both and can keep each in its proper field is certainly employing the art of bridge building to the best advantage.

By Joseph R. Worcester.—The interesting questions which have been raised by the discussions of the paper have shed light on a number of points which have not been sufficiently elaborated in the paper itself, and the author is very glad to acknowledge that many of the criticisms are well founded. There have, however, been a few points raised which perhaps can be explained.

With regard to Mr. Shaw's points about field rivets it appears to the author that the danger in this class of work is not so much from imperfect heads as from the fact that the holes are not completely filled. If such be the case, unless we are willing to allow considerable slip when the load is applied, it is essential that we should provide rivets enough to hold by friction. The head must be very bad if it is not sufficiently enlarged to grip the plates together when the rivet cools. In the author's opinion it is better to use bolts where good rivets cannot be driven, for a bolt well drawn up is much better than a loose rivet: one loose rivet in a joint is absolutely useless until the others have yielded.

So far as the action of repeated or reversed strains affect the question of friction, if we have no motion between the surfaces clamped together, we have a constant strain in the rivet. It is not apparent, then, how the rivet can be weakened by any number of variations of stress.

Of course, if there is any motion the strength becomes less with each slip and we soon have a loose joint. It is therefore all the more important to provide rivets enough at the start to prevent motion.

Mr. Howard's careful consideration of the effect of the friction is very instructive, and his explanation of the variation of the elongation during the earlier stages of the tests from that which would occur in solid metal appears quite reasonable. The author's only reason for suggesting that a less spacing than three diameters may not give as good results as a greater is from observation of the large number of Watertown tests made with all kinds of spacing. It certainly seems necessary that more tests should be made to definitely determine this point.

While the author agrees with Prof. Lanza that it is somewhat difficult to determine from the Watertown tests the point where a decided slip takes place, in most cases it is very evident that there is such a point, as may be seen by carefully plotting the extensions. The author cannot agree as to there being any danger in considering the friction in the method suggested in the paper, and it is not easy to see what is meant by a joint having more frictional resistance than strength. It is not, of course, proposed to govern the thickness of the plates or parts connected by the frictional resistance of the joint, as this element is invariably settled by other considerations.

With regard to the experiments of M. Dupuy, while the author does not quote them as being conclusive they seem to be particularly instructive on account of the thoroughness with which the minute elongations were observed under light strains and also on account of the care exercised in determining the physical conditions of the rivets.

The remarks of Mr. Moses with regard to the difficulty of securing an adequate number of tests must be apparent to all. It is very much to be hoped that in the future this subject will not be so neglected as it has been in the past.

The author is pleased to note the extent to which Mr. Snow has been willing to consider friction in his own practice.

In conclusion the author cannot but repeat what he has stated in the paper that while in good practice, that is, in work that is not strained above one-half its elastic limit, practically all riveted joints are held by the friction alone, it certainly seems wrong to consider as the basis of strength, elements, such as bearing, which cannot possibly come into play until the joint has yielded.

A LOW CRIB DAM ACROSS ROCK RIVER.

By J. W. Woermann, Member of the Engineers' Club of St. Louis.

[Read before the Club, May 6, 1896.*]

The dam which is herein described constitutes one of the structures of the Illinois and Mississippi Canal, better known as the Hennepin Canal, and was built for the purpose of furnishing slack water navigation in Rock River above the Lower Rapids, which are located at Milan, Ill. The location and general engineering features of this canal were briefly described in a paper devoted to the concrete construction on the same, read before the Club about two years ago, and published in the journal of the Association for November, 1894. Since that time the four and one-half miles of canal around the Lower Rapids, together with about eight miles of slack water above the dams, or about thirteen miles in all, have been opened to navigation, and some coal traffic is being developed. During the same time eight miles have been completed on the Eastern Section, immediately adjoining the Illinois River, including the masonry for seven locks, one aqueduct and a number of minor structures. The concrete abutments for this dam or dams, as there are two sections of it, one or either side of Carr's Island, were described in the former paper, so that only the cribwork or dam proper will be considered at the present time.

LOCATION.

The determination of the best location for the dams involved a complete survey of this vicinity, including not only ordinary soundings over a considerable stretch of the river, and the taking of topography along the shores, but also the preparation of profiles, showing the elevation of bed-rock at the most feasible sites. The topography along the shores was necessary, as some of the locations required a greater or less extent of levee to protect the adjoining lands from overflow.

The location finally selected requires a dam across the south channel of Rock River, at the head of Carr's Island, 764.2 feet in length, and another dam across the north channel 598.3 feet in length, about 800 feet below the head of the island, giving a combined length of 1,362.5 feet of crest. Connecting the two dams is a levee about 1,000 feet long to protect the island from overflow at high-water.

* Manuscript received July 20, 1896.—*Secretary, Ass'n of Eng. Socs.*

DESIGN.

The general design of the dam was prepared by Major W. L. Marshall, Corps of Engineers, U. S. A., the general plan of which can be most readily seen by a glance at Plate VI accompanying this paper. It was designed for a rock foundation to withstand a maximum head of four and a half feet, and may be described as a rock-filled crib, the woodwork consisting principally of six-inch by eight-inch pine timbers laid flatwise. The main part of the dam is thirteen and a half feet in width and the apron six and one-half feet, making the total width of the base twenty feet. Immediately adjoining the cribwork above is a filling of clay and quarry refuse of about the same width as the cribwork, and rising in height to the top of the sheet piling.

The main dam and the apron are both covered with four-inch oak plank, and the up-stream face of the dam with two rows of two-inch pine sheet piling. The oak plank on the main dam are closely fitted together, making it practically water-tight, so that the vertical pressure of the water above the coping is added to the weight of the material in the dam in giving increased stability to the structure.

The coping of the main part of the dam is built on a slope, rising two feet from the sheet piling to the crest, with a view of preventing projecting limbs and other irregular objects from getting caught on the up-stream face and pounding more or less upon the coping, as is usually the case where the slope is in the opposite direction.

From the crest of the dam to the apron is a fall of three feet. The top of the apron is about six inches above extreme low-water, but at the stage at which the ice usually goes out it is covered with water, more or less, forming a cushion which prevents the ice from cutting the apron as it otherwise would. In the spring of 1895 the ice went out at an unusually low stage, with a thickness of six to twelve inches, but did no damage. The six-inch by eight-inch transverse pine timbers, projecting two inches above the level of the apron, were cut down to the level of the oak plank to some extent, but this was anticipated and was not considered important.

NORTH COFFERDAM.

The construction of the dams was in charge of Mr. L. L. Wheeler, M. Am. Soc. C. E., as Resident Engineer, with the writer as principal assistant, and Mr. Geo. T. McGee as instrumentman. The best form of cofferdam to use in shutting off the river was a matter of considerable investigation, and the contingencies and probable cost were estimated for several different styles. Inasmuch as Rock River is usually at a low stage during the summer, and as the spring of 1894 had been unusually dry, it was decided to build a simple earth embankment across each

channel with riprapping on the up-stream side to protect them from wave-wash. This plan received greater favor also from the fact that the cofferdam around the guard lock had been successfully built in this manner, and because the bed of the river was of such a nature that teams could be driven over it wherever the depth was less than three feet. As the north channel contains the deepest water, the greatest depth at that stage being about five feet, it was decided to build that dam first, and leave half of it incomplete to serve as a temporary sluice-way during the construction of the South Dam.

An area below the north abutment of this dam was stripped in April to furnish a site for a quarry, and early in June the construction of the cofferdam was commenced. The stripping from the quarry, together with the quarry refuse, formed the body of the cofferdam, while a ridge of riprap was kept in advance on the lower side to prevent the current from washing away the loose earth. While the embankment was being started from the north shore of the river, five cribs, sixteen feet square, were settled in line adjacent to Carr's Island. The cribs were built in the shallow water near the shore, by simply boring a hole in each end of each timber and dropping them over the long bolts which held the timbers together at each corner of the crib. The cribs were placed fourteen feet apart, the top covered with four-inch oak plank, and weighted down with rock and bags of sand. Six by eight-inch timbers, the ends of which were supported by the cribs, were then shoved down into the water, furnishing a length of about 130 feet to sustain the riprap from being carried away by the current. The riprap and earth cofferdam was then extended to the island, above this protection, and the flow completely shut off. The weight was then taken off the cribs and the lumber used in the construction of the permanent dam. Subsequently the riprap was removed and used for filling in the permanent structure,—this recovery of the riprap being the principal argument in favor of placing the stone on the down-stream side of this cofferdam.

The end of the cofferdam was kept about two feet above the surface of the water, the wagons being dumped while they stood on the steep slope at the end. A small amount of riprap was placed on the up-stream side, above the water-line, to protect the embankment from wave-wash. The teams returned to the shore by driving through the water on the upper side of the cofferdam. On account of the south channel remaining entirely open, the construction of this cofferdam only raised the water surface about four inches. The foot of the island was far enough down-stream to keep the backwater from coming up and interfering with the work. Low secondary cofferdams, a few inches in height, were then built below the main cofferdam, to exclude the seepage that came from the latter. The areas enclosed were from fifty to two hun-

dred feet in length, according to the irregularity of the bottom, and were kept dry by means of hand-pumps. The entire amount of material in this cofferdam, including what was in the secondary dams, was about 300 cubic yards of riprap and 800 cubic yards of quarry stripping.

FOUNDATION.

All sand and gravel, together with as much of the bed-rock as could be readily raised with a pick, were then cleared away and the construction of the cribwork commenced. Where the rock was comparatively smooth and solid, iron anchor bolts were set in cement, in holes drilled for the purpose, to which the foundation timbers were bolted so as to insure a greater factor of safety against sliding. The bolts were usually one and one-eighth inches in diameter and twenty-four inches long, but where the bottom was less firm longer bolts were used. Two bolts were generally placed in each panel, the spacing depending on whether the first course consisted of longitudinal or transverse timbers. Where the rock was loose enough to permit a trench to be picked out, six inches or more in depth, for the base of the dam, the anchor bolts were considered unnecessary. The largest pocket of clay was ten feet across, and was excavated to a depth of six feet below the river bottom before starting the cribwork. Over the three largest pockets the apron was made fourteen feet in length instead of the regular seven-foot length.

CRIBWORK.

All the timbers in the dam were six by eight inches except the top timber on the up-stream face and the top timber under the crest, which were eight inches by eight inches, and eight inches by ten inches, respectively. All of the longitudinal timbers were sixteen feet in length and were arranged so as to break joint regularly and to bring the joints within two feet of the middle of the panels. The bottom course consisted of six rows of timbers, so as to furnish more area for the support of the rock filling, and each of the succeeding courses only five, up to the level of the apron.

In laying the bottom timbers readings were taken on them at frequent intervals with a wye level so as to insure their being started at the proper grade. Where the bottom was comparatively regular the carpenters extended the work several panels at a time with common spirit levels. Throughout the work the bottom timbers were adzed to fit the irregularities in the rock so as to insure greater safety against sliding. Readings were again taken on the cross-timbers at the level of the apron, and on the eight-foot blocks near the top, and wherever the elevations exceeded the proper grades by more than a half inch

the course next following was notched down accordingly. The differences in elevation were caused mainly by the variation in thickness of the timbers. The lumber being only regular commercial stock, the thickness varied all the way from five and a half to six inches. On this account also the bottom timbers were started about two-tenths of a foot above the elevation shown on the plan so that allowance could be made for this deficiency. An absolutely level crest at an exact grade, however, was not considered of sufficient importance to warrant much additional expense in notching down timbers, so that no great refinement was attempted in this direction. The highest and lowest points on the crest of the North Dam were 130.04 and 129.91 respectively, and for the South Dam 130.59 and 130.46 respectively. The maximum difference in each case is thirteen-hundredths of a foot, and the mean elevation determined from readings taken every sixteen feet are 129.992 and 130.530, Hennepin Datum.

From the level of the apron upward the transverse timbers are of different lengths in order to properly support and reinforce the purlins, as shown on Plates V and VI. The purlins were five in number, spaced about three feet and three inches between centers. The use of the templates in marking the gains to receive the purlins is shown graphically on Plate VI and requires no further explanation.

The transverse timbers are all spaced eight feet apart except that in the course above the bottom longitudinals an extra timber fourteen feet long is placed in the middle of each eight foot panel, so as to assist in receiving the weight of the rock filling. On the down-stream face of the dam, under the apron as well as under the coping, a two-foot block is placed under each joint, to which the longitudinals are thoroughly bolted. The intention was to increase the tensile strength on that side, so that in case any part of the dam should ever be called upon to act as a beam, it would have proportionately greater transverse strength.

DRIFT BOLTS AND SPIKES.

The method of drift bolting can be seen most readily by glancing again at Plate VI. The size of all the drift bolts was three-quarter inch by sixteen inch except that ten-inch bolts were used at the bottom wherever the first course happened to consist of longitudinals, and eighteen-inch in putting on the eight by eight-inch and eight by ten-inch longitudinals. One bolt was driven at each intersection, the bolt being always started through a cross-timber. Holes were bored to the full depth of the bolts one-sixteenth inch smaller in diameter.

The four-inch oak planking on the coping and apron was fastened down with two seven-sixteenths inch by eight-inch boat spikes at each purlin, for which seven-sixteenths inch holes were bored. The first row

of two-inch sheet piling was held temporarily with 20*d* wire nails until the second row was put on, both rows being carried along together, after which they were fastened permanently with four three-eighths inch by seven-inch boat spikes per running foot, driven without boring.

ROCK FILLING.

The filling of the dam was carried on simultaneously with the crib-work, and the stone packed in between the timbers so as to obtain as much weight as possible. The filling immediately adjoining the crib-work on the up-stream side was carried along in a narrow embankment as fast as the sheet piling was put on. This permitted the rock teams to be driven up close to the sheet piling, and made it possible to throw the stone directly into the cribwork from the wagons. When the dam was nearly in the condition in which it was to be left during the construction of the South Dam, this embankment was widened by casting over material from the cofferdam, until the latter was finally allowed to break through.

TEMPORARY SLUICEWAY.

The temporary sluiceway in the North Dam, previously referred to, kept the water above the dams about two feet lower than if the dam had all been completed at one time,—and allowed the South Cofferdam to be made lower by the same amount. The saving by this plan must not be measured directly by the difference in the volume of the South Cofferdam under the two conditions, but by the fact that the average depth of water in which it was necessary to work would have been increased from about two feet to four feet, in case the sluiceway had not been used.

A temporary bracket consisting of a vertical post and two braces was set up at each panel point, as shown on Plate V. On the up-stream side of the posts, near the top, was supported a line of four-inch by twelve-inch pine wales. This brought the wales on line with the permanent sheet piling, which formed the bottom of the sluiceway. In shutting off the sluiceway subsequently, in order to complete this portion of the dam, it was necessary simply to shove the sheet piling down in front of the wale, and allow them to catch on the top of the permanent sheet piling. The vertical posts and the long braces were used in the completion of that half of the dam, and the balance of the lumber on other parts of the canal, so that there was no waste of lumber. During the construction of the South Dam this sluiceway carried the whole discharge of the river, amounting to about 2,500 cubic feet a second.

SOUTH DAM.

The construction of the South Cofferdam was commenced on July 24th, and completed on August 4th. In this case the ridge of riprap was

extended ahead of the earthen portion on the up-stream side, so as to permit the teams to return to the shore on the lower side, as the increased depth, caused by shutting off the water completely, would not permit the same plan to be used as at the North Dam. On account of the greater amount of water with which it was necessary to contend in making the final closure, ten cribs were erected adjacent to the Island instead of five. These were settled on the up-stream edge of the cofferdam. When the end of the cofferdam had been extended so as to come under the protection of the first crib, wales were placed across the spaces between the cribs and driving sheet piling was commenced from both ends of this three-hundred foot space covered by the cribs. Under this protection work on the rock and earth cofferdam was also carried on from both ends. As the ends of the cofferdam were extended part of the sheet piling was taken up and used a second time, and only in making the final closure for the last hundred feet was it necessary to double the piling. The method of dumping the wagons and the final closing behind the cribs are shown on Plate I.

When the comparatively tight embankment had been completed the weight was taken off the cribs and the lumber used in the permanent structure. Most of the clay and riprap for this cofferdam were taken from a waste pile of material that had been excavated from the lock pit at the south end of the dam.

As indicated by the borings, it was found that the foundation of this dam was not as good as the other, inasmuch as a stretch of hard clay was encountered 50 feet in length, and a bed of compact sand and gravel 120 feet in length. This was excavated to a depth of about three feet below the bed of the river, and the material used as filling above the dam. As the cribwork progressed, the V-shaped trench that remained below the apron was filled with heavy stone, as shown on Plate V.

The carpenter work on this dam was commenced on August 7th and completed August 22d, sixteen days from the time the first timber was laid. The Federal labor law, passed by Congress in 1892, prohibited us from working more than eight hours a day, without putting on a second shift, which was considered impracticable, but it did not prevent us from working on Sundays.

The construction of this cofferdam raised the water surface about two feet higher than it was during the building of the North Dam, and in order to provide a sluiceway for carrying the seepage from the cofferdam through the permanent dam, the sheet piling was omitted from one panel until the filling above the dam was all completed. The cofferdam was allowed to break through on August 24th, exactly one month from the time the cofferdam was commenced.

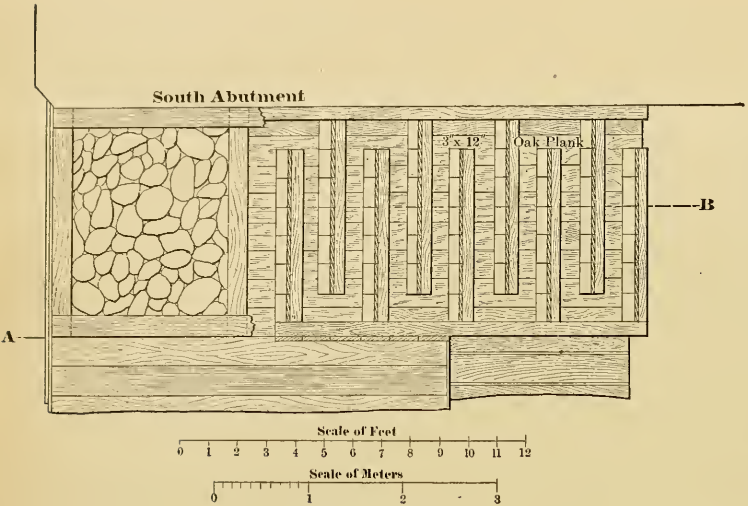
The rock filling for this dam was obtained considerably cheaper than

Plate I.

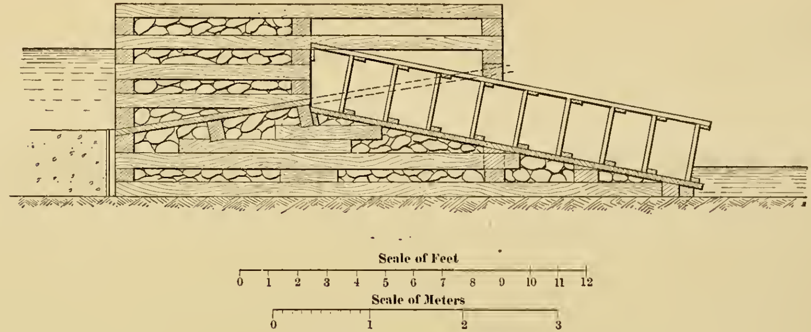
Plate II.

PLATE III.

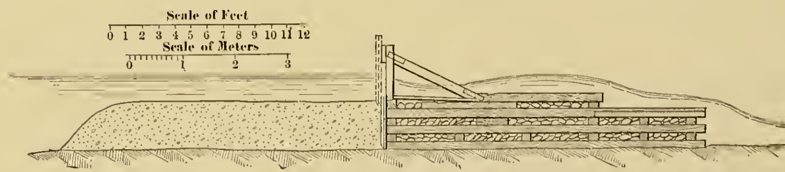
Plate IV.



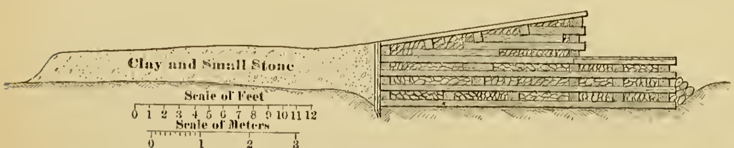
Plan of Fishway.



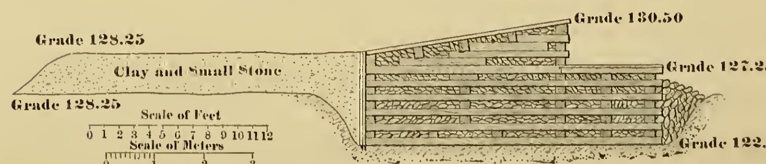
Section A-B of Fishway.



Temporary Sluiceway in North Dam.



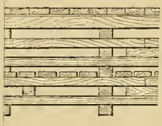
Typical Section on Rock Foundation.



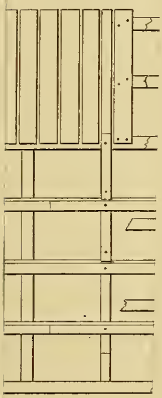
Typical Section on Gravel Foundation.

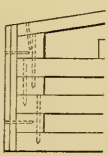
PLATE V.

EERING SOC

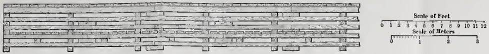


Elevation of D

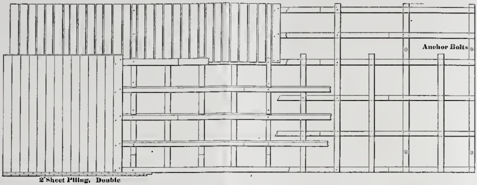




P



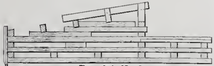
Elevation of Down-Stream Side.



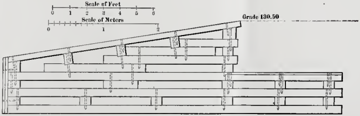
Plan.

Cross-Section.

Template No. 1



Template No. 2



Cross-section.

PLATE VI.

for the other from the fact that on this side about 75 per cent. of the rock was readily quarried from the bed of the river without explosives.

FORCE EMPLOYED.

During the construction of the South Cofferdam the force consisted of about fourteen teams and fifty laborers. For a few days while the preparation of the foundation was being rushed the number of laborers was increased to 130.

During the erection of the cribwork the force consisted of sixteen carpenters, and about fifty laborers, about one-third of the latter assisting the carpenters in carrying timbers, boring, and driving bolts and spikes. The number of teams remained the same throughout the work. The appearance of this dam during construction is shown on Plate II, while the view presented with the water running over it, after both dams were completed, is shown on Plate IV.

COMPLETING NORTH DAM.

After the completion of the South Dam the temporary sluiceway in the North Dam was closed as previously described, and the upper part of the dam completed readily under the protection of the sheet piling. As soon as the sheet piling was in place the lower braces were knocked out, so that there was nothing to interfere with putting on the purlins. As the coping was gradually extended from the abutment; braces were put in from the waling-piece back to the top of the oak plank. This permitted the post and the remaining brace to be knocked out, as the spikes in the sheet piling were sufficient to support the weight of the waling-piece. . The stone filling was thrown into the crib from a barge which was towed alongside the sheet piling. The completion of this part of the dam occupied four and a half days, including the erection and removal of the sheet piling. Plate III was taken during the completion of this portion of the dam.

FISHWAYS.

At the time the dams were built a fishway was constructed at the south end of each dam. It was found, however, that they were unsatisfactory for several reasons, and during the following summer they were modified according to the plan shown on Plate V. By increasing the number of wings from five to nine the velocity of the water was checked so that fish can readily ascend from step to step. The upper end is arranged so that the fish go out into comparatively quiet water, instead of having to jump over the crest, while at the same time the amount of water entering the fishway can be regulated to suit the stage of the river. They also comply with the theory that a fishway, in order to be

found readily by the fish, should not extend down-stream any farther than the apron of the dam. The cribs above the fishways, together with their protected position adjacent to the south abutments, are designed to protect them from ice. When the fish are running up-stream the larger ones can frequently be seen entering and leaving the fishways. By shutting off the water at the upper end, as many as sixty fish of various species have been found in it at one time, some of which have been from two to three feet in length.

TOTAL AMOUNT OF MATERIAL IN DAMS.

	North Dam.	South Dam.
	Ft. B. M.	*Ft. B. M.*
Longitudinal timbers	47,230	73,550
Transverse "	28,350	46,950
Sheet piling	7,950	14,610
Total pine lumber	83,530	135,110
Oak plank in coping	33,540	42,840
" " " apron	15,870	19,300
Total oak lumber	49,410	62,140
Total oak and pine lumber	132,940	197,250

The total amount of lumber in both dams is 330,190 feet, B. M. The cost of the labor expended in putting this in the dams amounted to $1,914, or $5.80 per M.

The total amount of rock filling in the North Dam is 1,240 cubic yards, and in the South Dam 2,350 cubic yards, making the total for both dams 3,590 cubic yards.

AMOUNT OF IRON IN DAMS.

	North Dam.	South Dam.
Anchor bolts	1,010 pounds.	320 pounds.
Drift "	6,050 "	9,610 "
Boat spikes	4,750 "	6,050 "
Wire nails	300 "	400 "
Total	12,110 pounds.	16,380 pounds.

COST OF LABOR ON EACH DAM.

	N. Dam.	S. Dam.
Hualing material		$283 97
Building cofferdam	$729 55	1,055 34
Preparing foundation	493 30	818 04
Carpenter work on dams	948 92	964 86
Quarrying rock, filling cribs, and grading above dams	1,965 54	1,970 56
Engineering, watching, and miscellaneous	362 25	402 47
Total	$4,499 56	$5,495 24

making the total cost of the labor on both dams practically ten thousand dollars.

TOTAL COST OF THE TWO DAMS.

The total cost of the two dams, including labor and material, is as follows:

Rent of land	$ 217 40
Labor	9,994 80
Oak lumber	2,919 00
Pine lumber	3,086 60
Explosives	151 19
Drift bolts, spikes, etc.	804 98
Total	$17,173 97

The total length of the two dams being 1,362½ feet, makes the cost per lineal foot $12.60.

Bradley & Poates, Engr's, N.Y.

Editors reprinting articles from this journal are requested to credit both the JOURNAL and the Society before which such articles were read.

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EXPERIMENTS ON VITRIFIED PAVING BRICK.

BY F. F. HARRINGTON, MEMBER OF THE ENGINEERS' CLUB OF ST. LOUIS.

[Read before the Club, June 17, 1896.*]

ABOUT this time last year Prof. H. A. Wheeler delivered before this Club an interesting lecture on "Vitrified Paving Brick," in which the processes of manufacture, the methods of testing, and the uses of paving brick were very thoroughly described. The speaker reviewed the methods of testing and the results of many experimenters, and explained many ways of combining the results of the various tests in order to show the relative merits of different kinds of paving material. Although the tests given were made by reliable men, the chief objection to them as a whole is that the methods employed were so different that the results, from a scientific standpoint, are inconclusive and not comparable. Only a short time ago, also, Prof. J. B. Johnson addressed the Club on "The Resistance to Crushing of Brittle Materials," including vitrified brick, and in this address the results of tests by Bauschinger, at Munich, were exhibited and many interesting relations shown.

About a year ago the Water, Sewer and Street Commissioners of this city established a testing laboratory, where all the material used by the city could be tested, instead of each department having its own testing done, as was formerly the custom. The use of vitrified brick had already been resorted to in the construction of alleys in the city, and its extensive use for street paving was about to follow. The question of determining the quality of the material therefore became an

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important one, and the laboratory was equipped with suitable apparatus for some of the essential tests.

A review of the past will show that vitrified paving brick has been used for several years in many cities and towns in the United States, and in most cases no laboratory tests were made, while in other cases reports of tests show them to be extremely crude and variable. Take for example the abrasion test. The material of construction of the rattler may be cast or wrought iron, steel, wood, or some combination of these materials; the size ranges from 12 inches to 60 inches in diameter; the speed from 200 to 10 revolutions per minute; the charge of bricks for the test varies from 5 to 40, usually composed of samples of different manufactures; sometimes whole bricks are used, and at other times cubical specimens are taken from them for test; the size of the abrading material ranges from that of shot to scrap iron pieces weighing over five pounds each; sometimes billets or blocks of wood are used as cushions for the bricks, and frequently granite, trap rock and other kinds of stone are introduced to furnish a comparison between brick and other paving material. The duration of the test varies from twenty minutes to three hours, and often an intermediate weighing is made, the loss during the first period of the rattling process, when the edges are defaced, being neglected, and that during the second period being taken for the loss due to abrasion. Specifications in different localities are therefore variable in their requirements and show a lack of information on the subject. Thus, in some cases, a few special tests must be employed, and in others, different ones, but it is very seldom that well-defined requirements are stated covering the essential tests. The paving-brick industry is now, however, developing rapidly, and the material is being extensively used in the residence districts of many of the largest cities, so the importance of an investigation of this subject is therefore apparent.

In 1895, at its annual convention, the National Brick Manufacturers' Association appointed a committee for the purpose of recommending for adoption a standard series of tests on vitrified paving brick and the methods of making them. This Commission is now actively engaged in the investigation of the subject. The first meeting was held August 1, 1895, at which time a preliminary standard method of making the various tests was agreed upon as a basis of conducting the necessary experiments. The experimental work was also apportioned between the members, in order to make the greatest haste in the preparation of the final report. The Commission is composed of engineers and brick manufacturers, and its recommendations, it is expected, will therefore be generally accepted.

It is intended to examine vitrified paving brick in this laboratory by means of the following tests: The rattler, absorption, cross-break-

ing, crushing, freezing, specific gravity and hardness. New machinery is being obtained for making all these tests. The Water Commissioner, Mr. M. L. Holman, has designed and built a new hydraulic testing machine for cross-breaking brick, having a capacity of 20,000 pounds, and has just completed the design of an hydraulic machine for crushing brick and other materials of construction, having a capacity of 1,500,000 pounds. This will be the most powerful crushing machine in the West.

The following are the results of the rattler and absorption tests made up to this time:

IMPACT AND ABRASION TEST.

The rattler test is considered by most engineers to be the most important test for vitrified brick. It is a measure of the toughness of the material, when laid in the pavement, to withstand the blows of horses' hoofs and the wearing action of the wheels of vehicles.

The tumbling barrel used for making the following tests is made of cast iron. It is polygonal in form, having 15 staves, and its dimensions are approximately 24 inches in diameter and 42 inches long. It revolves on trunnions. Within the barrel is a cast-iron partition at right angles to the axis by means of which the length can be varied. The barrel is operated by a constant speed electric motor, through a main shaft, counter-shaft and gear wheels.

The vitrified paving brick used for the tests were made of pure shale, worked by the stiff mud process, and burned in down-draught kilns. Five hundred well-burned samples were selected from one kiln, with the view of obtaining the most uniform specimens possible. They are re-pressed brick and have rounded edges. The dimensions are 8¼ x 4 x 2½ inches. The volume of one brick is therefore 82½ cubic inches, and the weight is slightly less than seven pounds.

The results of the tests are shown in Figs. 1, 2, 3 and 4.

Fig. 1.—An arbitrary length of barrel of 30 inches and speed of thirty revolutions per minute were chosen to begin the work. Five per cent. of the volume was then filled, requiring eight bricks, and the percentage of loss calculated after forty and eighty minutes' tumbling. In the same manner the barrel was filled to 10, 15, 20 and 25 per cent. of its volume, using 16, 24, 32 and 40 bricks respectively, and the percentages of loss calculated as before, after forty and eighty minutes' tumbling. The maximum percentage of loss was thus found to be when the barrel was filled to 15 per cent. of its volume.

Fig. 2.—For the experiments shown in Fig. 2, the cast-iron partition in the barrel was moved so as to make the length successively 12, 21, 30 and 39 inches, and for each test 15 per cent. of the respective volumes were filled, requiring 10, 17, 24 and 31 bricks for the charges.

The percentage of loss was calculated in each case after tumbling forty and eighty minutes. It will be seen that the percentage of loss is practically independent of the length of the barrel, when 15 per cent. of its volume is filled.

Fig. 3.—For these tests a length of barrel of 30 inches was chosen, as in the tests in Fig. 1, and 15 per cent. of the volume of barrel, or twenty-four bricks, were tumbled in each experiment. The barrel was run at speeds of 20, 25, 30, 35 and 40 revolutions per minute by changing the pulley on the main shaft, and the percentage of loss was calculated after 10, 20, 30, 40, 60 and 80 minutes' tumbling for each test. It will be seen that the percentage of loss continues to increase with the increase of speed and at a more rapid rate. This evidently shows that the construction of the barrel is such that a speed of forty revolutions per minute is not sufficient to cause the bricks to revolve around the axis of the barrel, nor is it sufficient to carry the bricks up to a height that will produce the greatest loss from the impact of the bricks upon one another, due to their fall. The curves indicated by 20, 30 and 40 minutes' tumbling are quite regular, while those of 10, 60 and 80 minutes' tumbling are irregular. The irregularity in the case of 10 minutes' tumbling may be accounted for from the fact that the edges of the specimens are being knocked off during this time, while that of 60 and 80 minutes may be caused by the disintegration of the bricks from the length of time that they have been subjected to the test.

Fig. 4.—In Fig. 4 the same tests are recorded as those shown in Fig. 3. The abscissa is here changed, however, to time of tumbling, while the speeds in revolutions per minute are written on the curves. These curves may be called "characteristic rattler curves" for the material at different speeds. They show the greatest loss to be during the beginning of the tests, when the edges are being defaced, and in general the percentage of loss is less with equal successive intervals of time during the continuance of the tests.

In making the preceding experiments, the bricks for each charge were weighed in bulk on a scale reading from ¼ pound to 250 pounds. The time of tumbling for all the tests was observed to the second. The speed of the barrel was so controlled that it did not fluctuate from that recorded a single revolution during the progress of the work. The most striking feature noticed was the uniformity of the brick tested. Thus the total weight of any 24 of the 500 bricks weighed, taken at random, did not vary more than a half pound. It was also found that the weight of any ten bricks did not vary more than one pound from that of any other ten bricks, when taken from the rattler at the completion of any particular test. This not only shows the brick to be very uniform, but also that the rattler is constructed so as to give results that are strictly uniform and comparable.

The results are:

(1) The maximum percentage of loss is obtained when the barrel is filled to 15 per cent. of its volume with brick.

(2) The percentage of loss is independent of the length of the barrel, when the foregoing condition is fulfilled.

(3) The percentage of loss increases at a more rapid rate than the speed from twenty to forty revolutions per minute.

(4) The percentage of loss decreases with equal successive intervals of time up to eighty minutes' tumbling.

A study of the results leads to the following recommendations for a standard rattler test.

Figs. 3 and 4 show that it is necessary first to choose a definite speed for running the barrel. A suitable speed would be thirty revolutions per minute, since this gives about the average circumferential speed of rattlers in general use. Then obtain characteristic rattler curves as shown in Fig. 4 for brick of each manufacture at the above mentioned speed. When these characteristic curves are drawn, it will only be necessary to find the percentage of loss after tumbling samples a given time for the test, or in other words to determine a single point on the curves. A suitable length of time for continuing this test would be forty minutes, since from Fig. 4 it appears that the edges of the brick suffer the greater loss from impact during the first twenty minutes, while during the next twenty minutes the exposed surfaces are abraded.

Having decided upon a definite speed and a definite time of tumbling, any one of the following three methods may be chosen for the standard test:

(1) Fill the barrel to 15 per cent. of its volume with the brick to be tested, adjust the partition in accordance with the number of brick on hand and tumble at the adopted speed for the chosen length of time. For this method it would be advisable to always test about fifteen bricks and move the partition according to their size. Not less than ten bricks should be tested.

(2) Clamp the partition at a definite position in the barrel, fill to 15 per cent. of its volume with the brick to be tested, and tumble at the required speed for the chosen time. For this method a good position for the partition would be in the middle of the barrel, making two chambers 21 inches in length, so that two tests could be made simultaneously. To make the test in this way, it would be necessary to use about seventeen bricks of standard size ($8\frac{1}{4}$ x 4 x $2\frac{1}{2}$ inches) or about thirteen of block size (9 x 4 x 3 inches).

(3) With partition clamped as in (2) put in the rattler the five or ten sample bricks to be tested and fill to 15 per cent. of volume of chamber with a selected uniform standard brick, kept in the testing

laboratory for that purpose, and tumble at the required speed and length of time.

RATTLER EXPERIMENTS WITH CAST-IRON BLOCKS.

A method of making the rattler test prevalent in some places has been to tumble five sample bricks with ten cast-iron blocks, each weighing about six pounds, for thirty minutes and determine the percentage of loss. In order to examine the reliability of this test, fifteen "unit bricks" as described above, were selected, five of which were tumbled with ten cast-iron blocks three successive times under the above conditions. The length of the barrel was 21 inches and the speed thirty revolutions per minute. Fig. 5 shows the results, the numbers on the curves giving the order of the tests. It will be seen that this method gives no characteristic curve such as we obtained when only bricks were rattled, and it is therefore apparent that the method represented in Fig. 5 is not a good one.

THE ABSORPTION TEST.

The absorption test is considered a very important one for paving brick for the reasons:

(1) For any particular brick, the percentage of absorption is an index to the degree of its vitrifaction.

(2) From a sanitary standpoint, it indicates the relative avidity for the retention of refuse matter, the evaporation of which pollutes the atmosphere with noxious gases.

(3) It furnishes a means of determining the possibility of the disintegrating action of frost.

The oven for drying the brick was designed for the purpose. It is made of galvanized iron lined throughout with asbestos. It is divided by a partition in two apartments, each 15 inches wide, 30 inches high, and 26 inches deep, and alike in every respect, so that only one side of oven need be described. There are four sliding grates, each holding fifteen standard size bricks, or sixty in all, and the full capacity of oven is 120. The heat is supplied by a Bunsen burner placed in the center of the bottom, the mixture of air and gas passing through numerous small holes of a special cap on the burner, and the flame impinges on an iron plate below the grates, thus heating the oven uniformly. The temperature is regulated by a damper in the flue, and read on a thermometer in the top of the oven.

The results of the drying tests are shown in Figs. 6 and 7. The temperature of the oven varied from 220° to 240° F. The makers of the bricks tested are designated by letters on curves, as follows:

(*a*) Alton Paving Brick Co., Alton, Ill.

(*b*) St. Louis Pressed Brick Co., Glen Carbon, Ill.

(*c*) Standard Paving Brick Co., St. Louis, Mo.
(*d*) Purington Paving Brick Co., Galesburg, Ill.
(*e*) Barr Clay Co., Streator, Ill.
(*f*) Townsend Paving Brick Co., Zanesville, O.
(*g*) Moberly Brick, Tiling and Earthenware Co., Moberly, Mo.
(*h*) Galesburg Paving Brick Co., Galesburg, Ill.
(*k*) Galesburg Brick and Terra Cotta Co., Galesburg, Ill.
(*l*) Royal Paving Brick Co., Canton O.

The average of two bricks of each kind was obtained for the curves: For Fig. 6, the bricks, as received from the makers, were dried in the oven for one week. For Fig. 7, these same bricks, when taken from the oven, were immersed in water twenty-four hours, and the drying process repeated for the same length of time. These two series of tests, it is thought, cover the conditions of the state of moisture of brick likely to be received at the laboratory for tests.

Figs. 8, 9, 10, 11, and 12 show results of the absorption tests. In Fig. 9, the absorption curves of Fig. 8, for the first three days in water, are shown on a larger scale. The figures on the curves of the plates represent bricks from the following manufacturers:

(1) Alton Paving Brick Co., Alton, Ill.
(2) St. Louis Pressed Brick Co., Glen Carbon, Ill.
(3) Standard Paving Brick Co., St. Louis, Mo.
(4) Purington Paving Brick Co., Galesburg, Ill.
(5) Barr Clay Co., Streator, Ill.
(6) Wabash Clay Co. (Poston Block), Veedersburg, Ind.
(7) Des Moines Paving Brick Co., Des Moines, Iowa.
(8) Townsend Paving Brick Co., Zanesville, O.
(9) Mack Paving Brick Co., Pittsburgh, Pa.
(10) Moberly Brick, Tiling and Earthenware Co., Moberly, Mo.
(11) Imperial Paving Brick Co., Canton, O.

Three uniform samples of each of the above kinds of brick were selected. They were dried in the oven for forty-eight hours. Two bricks of each kind were immersed whole. The results are shown in Figs. 8 and 9. Both ends of the third brick of each kind were removed, leaving about half bricks with two surfaces from interior exposed to absorb water. These results are shown in Fig. 10. Also, a small piece from the interior of the third brick of each kind, weighing about 25 grammes, was tested, the results being shown in Fig. 11. The temperature of the water in which the bricks were immersed averaged about 60° F. The water on the surface of each specimen was removed with a dry cloth before weighing. The whole and half bricks were weighed on a balance to the nearest gramme. The small pieces were weighed on a chemical balance.

RESULTS OF ABSORPTION TESTS.

Figs. 6 and 7 show that it requires four days to thoroughly dry vitrified brick when subjected to a temperature ranging from 220° to 240° F. under all usual degrees of moisture, and that in forty-eight hours they are practically dry. Thus, 94.1 per cent. of the whole amount from the bricks in normal state of moisture was evaporated in two days, and 95 7 per cent. of the whole amount contained in the samples previously immersed for twenty-four hours was driven off in two days.

Figs. 8 and 10 show that both whole and half bricks continue to absorb water up to twenty-four weeks, at which time the experiments were discontinued. For practical purposes eight weeks would be required to soak them. The percentage of absorption of the half bricks ordinarily exceeds that of whole bricks of the same manufacture. Thus at twenty-four weeks' immersion the percentage is greater for half bricks than for whole ones, except in numbers 3, 6 and 10. The average increase of half over whole bricks after soaking twenty-four weeks is 16.5 per cent.

Fig. 11 shows that the small pieces continue to absorb up to eight weeks, and also that the percentage of absorption of small pieces in eight weeks exceeds that of half bricks of the same manufacture except in the case of number 8, and that of whole ones of Fig. 8 without any exception. The average increase of small pieces over half bricks in eight weeks is 47.3 per cent , and over whole ones in the same time 66.1 per cent.

Fig. 12 shows the average absorption curves of whole bricks, half and small pieces found from Figs. 8, 10 and 11.

A study of the figures suggests the following method for a standard absorption test. Let about ten samples of the bricks to be tested be dried in the oven for at least two days. Then immerse in clear water and obtain characteristic absorption curves for each manufacture as in Fig. 8 for a length of time of eight weeks. Rattled bricks should be used for these tests, since the preceding experiments show that a higher percentage of absorption is obtained when surfaces from the interior are exposed. Furthermore, this method conforms better with the conditions of actual service, since it is only a short time after the bricks are laid in the pavement before some of the exposed surfaces are worn away.

When the characteristic absorption curves of various kinds of brick have been obtained, it will only be necessary to immerse the samples, the characteristic absorption curve of which is known, for any convenient length of time, and obtain the percentage of absorption for that time.

PLATE I.

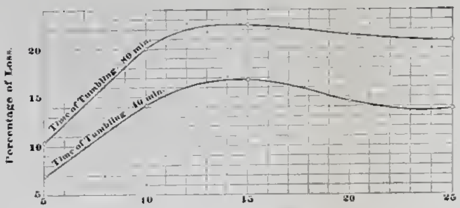

FIG. 1.

Diameter of barrel, 24 inches.
Length of barrel, 30 inches.
Revolutions per minute, 30.

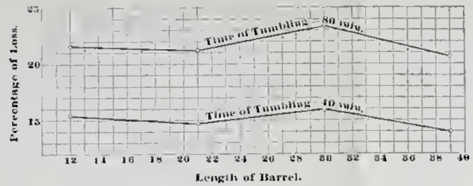

FIG. 2.

Diameter of barrel, 24 inches.
Revolutions per minute, 30.
15 per cent. of volume of barrel filled with unit brick.

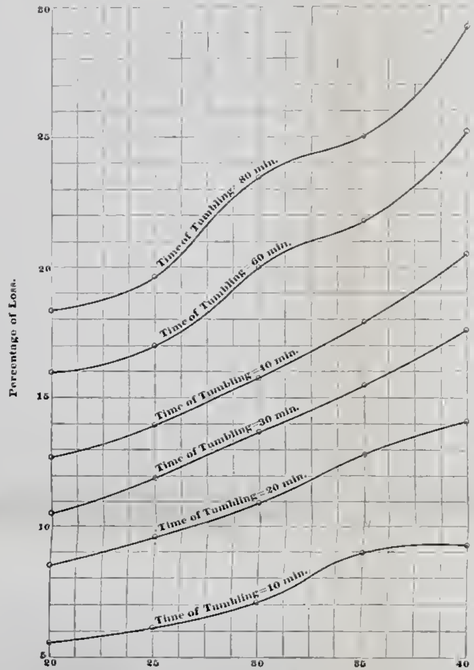

FIG. 3.

Diameter of barrel, 24 inches.
Length of barrel, 30 inches.
15 per cent. of volume of barrel filled with unit brick.

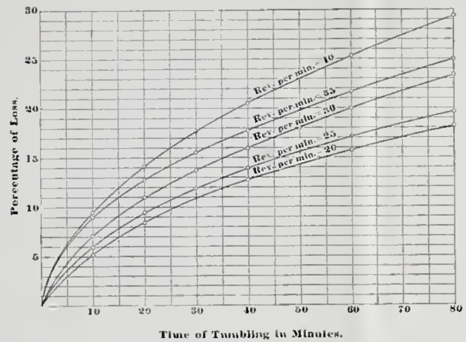

FIG. 4.

Diameter of barrel, 24 inches.
Length of barrel, 30 inches.
15 per cent. of volume of barrel filled with unit brick.

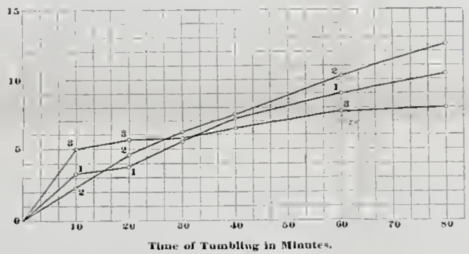

FIG. 5.

Experiments with cast iron blocks in rattler. Brick and 10 cast iron blocks.
Diameter of barrel, 40 inches.
Length of barrel, 21 inches.
Revolutions per minute, 30.
Charge for each test, 5 unit.

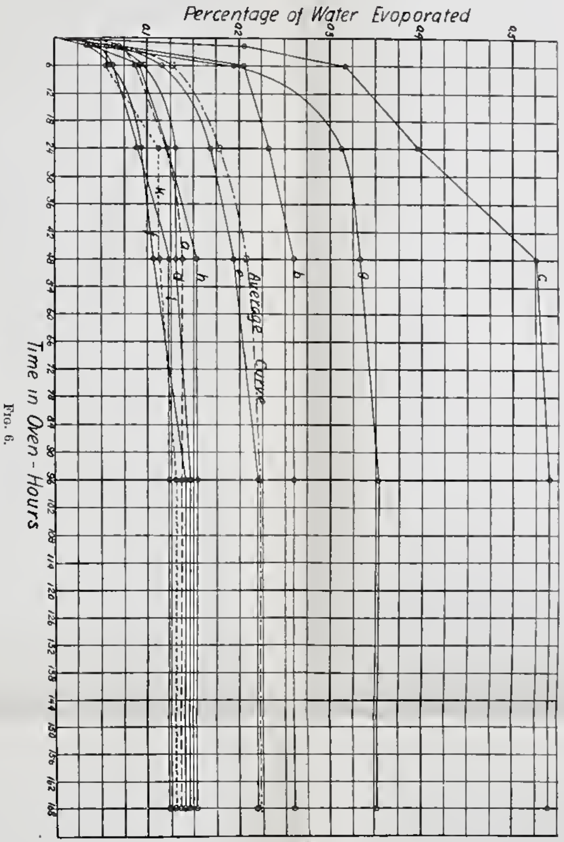

FIG. 6.

Drying Tests. Normal moisture. Each curve is the average of two bricks. Reference letters marked on curves.
Temperature of oven, 220° to 240° F.

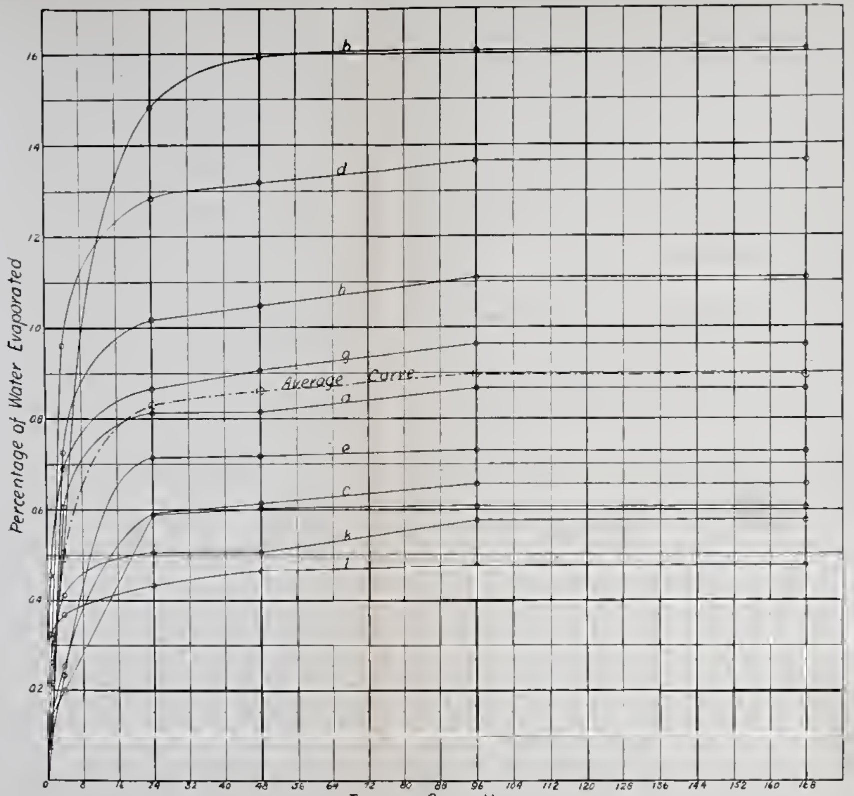

FIG. 7.

Drying Tests. Immersed in water twenty-four hours. Each curve is the average of two bricks. Reference letters marked on curves. Temperature of oven, 220° to 240° F.

PLATE II.

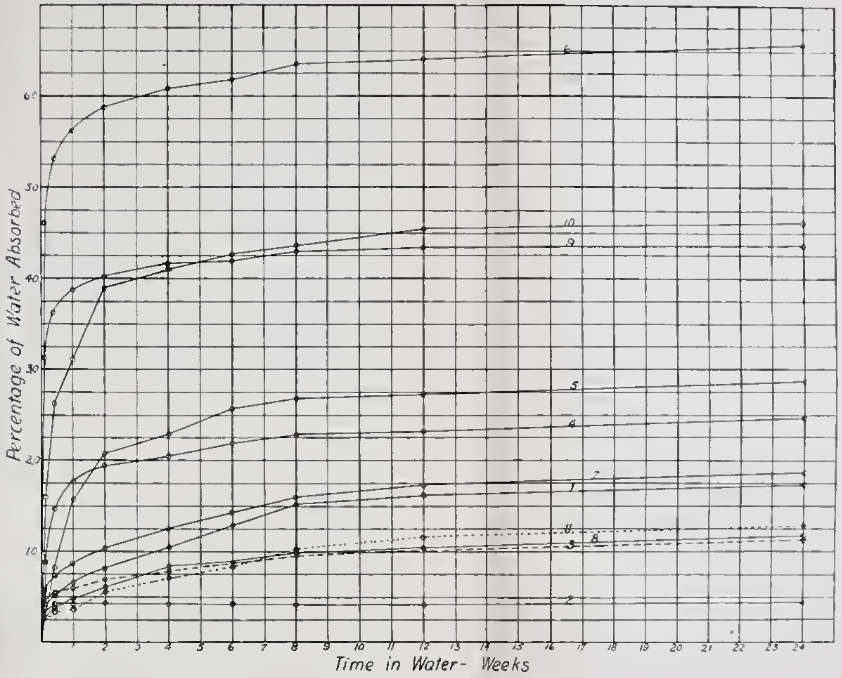

FIG. 8.

Absorption Tests. Whole bricks tested. Each curve is the average of two bricks. Reference numbers marked on curves.

PLATE III.

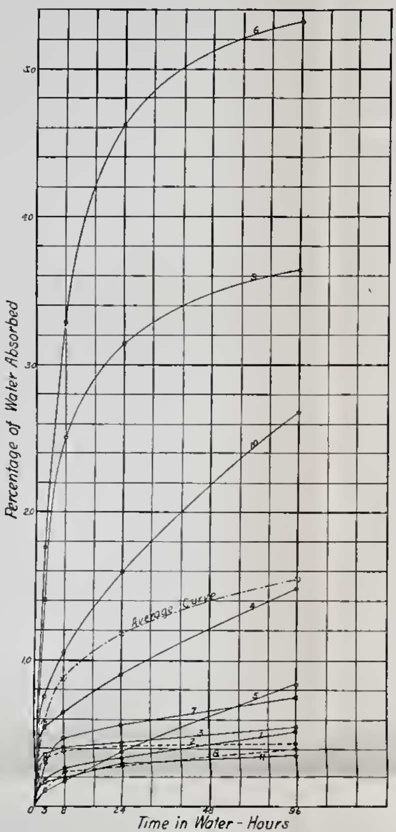

FIG. 9.

Absorption Tests. Whole bricks tested. Each curve is the average of two bricks. Reference numbers marked on curves.

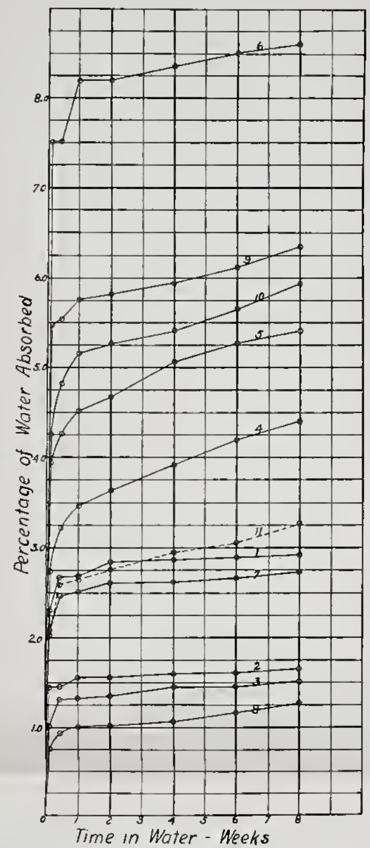

FIG. 11.

Absorption Tests. Small pieces from interior tested. Each curve represents one piece. Reference numbers marked on curves.

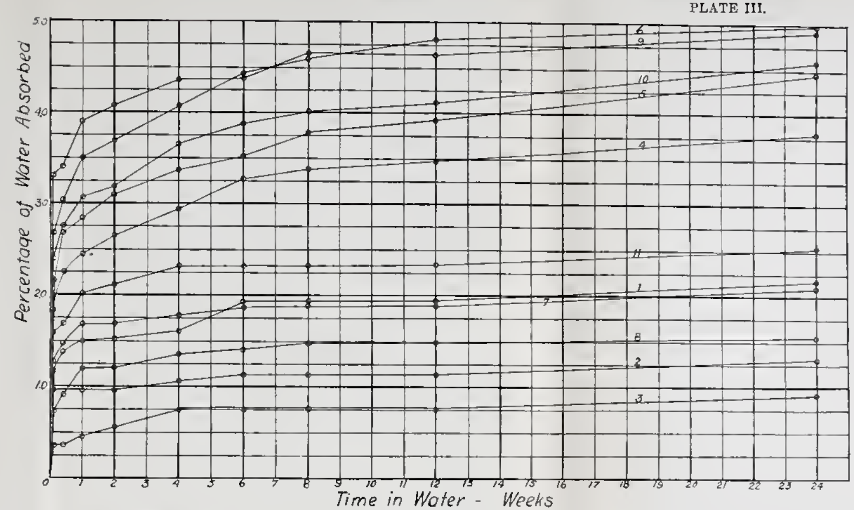

FIG. 10.

Absorption Tests. Half bricks tested. Both ends of each brick removed, leaving two surfaces from interior exposed. Reference numbers marked on curves.

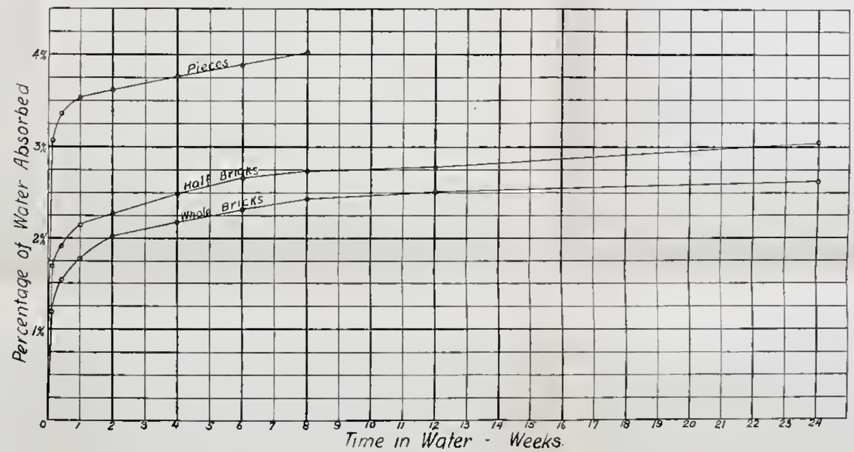

FIG. 12.

Absorption Tests. Resulting average curves of whole bricks, half bricks and small pieces.

THE CONDITIONS NECESSARY FOR EQUALITY OF VELOCITY IN PARTICLES SETTLING THROUGH LIQUIDS.

By Luther Wagoner, Member of the Technical Society of the Pacific Coast.

[Read before the Society, August 7, 1896.*]

Under the title "On the maximum velocity acquired by small bodies falling in water and glycerine," the writer published a paper in *Proceedings Tech. Soc. Pac. Coast*, March, 1888, wherein certain conclusions and empirical formulas were presented differing from the views previously held, and as the question is one of practical importance, especially in the dressing of ores and the separation of bodies by air or water, the writer has been induced to take up the subject again.

The published results of Prof. Richards, A. I. M. Engrs., Vol. xxiv, furnish data much superior to anything previously had, and, as his paper may easily be found, only a short abstract of his methods will be given. Thirteen kinds of minerals were experimented with. They were first assorted upon sieves into different sizes ranging from 10–12 to 120.140 mesh. The diameter of each sieve aperture was carefully measured and the diameter of the ore grain is taken as a mean between the sieves passing and those rejecting the grain. Fifty grains or particles of sized mineral were next dropped into a vertical glass tube, and the time required for 90 per cent. of the grains to pass two wires eight feet apart gives data for finding the mean velocity; the experiment was repeated ten to twenty times for each size, and the mean for all was adopted. We thus have data connecting the diameter or mean sieve opening and the maximum velocity of fall in the water. To have been complete the data should have given the average weight of a grain of each mineral. The immediate object of this discussion is to examine the facts about grains under one millimeter size, and the data of Prof. Richards has all been reduced from inches to millimeters, the *m.m.* being taken as unit for diameter x. and velocity equal v.

METHOD OF DISCUSSION.

Referring to Fig. 1, where the diameters x of the grains are shown as abscissæ and the velocities v as ordinates, it is required to find an equation connecting v with x and which will be reasonably correct for diameters

* Manuscript received August 13, 1896.—*Secretary, Ass'n of Eng. Socs.*

smaller than the lowest values of $x = 0.1171$ mm. or from $x = 0$ to $x = .1171$. The impelling force is gravity and the weight of the body is a function of its diameter. The retarding forces are the cross-sectional area and perhaps the surface, but both are functions of the square of the diameter. The equation for uniform motion may be written

$$v = k\sqrt{\frac{x^3}{f\,x}} \qquad \text{(Eq. 1)}$$

where v = velocity in mm., x = diameter in mm. of the opening in the

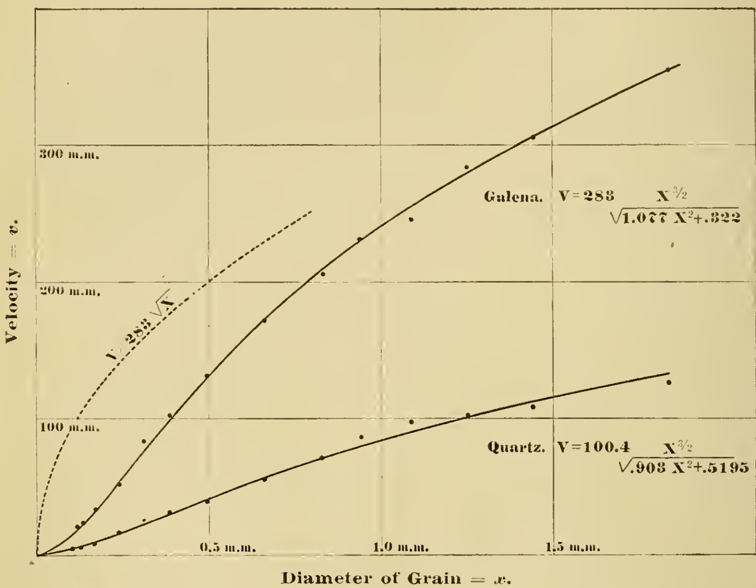

Fig. 1.

sieve. k is a general coefficient and $f.\,x$ is considered the unknown quantity and is found

$$f.\,x = \frac{k^2\ x^3}{v^2} \qquad \text{(Eq. 2)}$$

v and x being known, k has been found as follows: Assume the various values of k to be proportional to the area of the curve shown in Fig. 1, or, what is the same,

$$\frac{k}{k_{\prime}} = \frac{\text{sum of } v}{\text{sum of } v_{\prime}}$$

A reduction of the experiments of Pernolet (vide *Annales de mine*, 1853, p. 144,) on coal, quartz, and galena 3 mm. to 30 mm. diameter combined with some experiments of the writer, gives the value of k for galena where x is the sieve aperture, equal $k = 283$, from which

all the other values of k for the different minerals become known. Substituting the proper value of k and solving the equation (2) for its 14 diameters, there résult 14 values of $f. x$. Several formulas were tried to find an equation for the denominator, and the simplest one $f. x = (a x^2 + b)$ was adopted. a and b were found as follows: let s and $s_{\prime}$ be the sums of the first and second sets of seven of x^2, and F and $F_{\prime}$ the corresponding sums of $f. x$, then

$$\begin{array}{l} a s + 7b = F \\ a s_{\prime} + 7b = F_{\prime} \end{array} \text{ and } a = \frac{F - F_{\prime}}{S - S_{\prime}}$$

This method of treatment gives equal weight to each of the observations. The general result of the investigation points to an increase in value of b for diameters below 0.2 mm., probably of the form $\frac{B}{C + f x}$. But as the data relating to form, surface and weight, in terms of diameter, are lacking, it is useless to attempt more approximate formula.

FINAL EQUATION.

$$v = k \frac{x^{\frac{3}{2}}}{\sqrt{a x^2 + b}}$$

making x large, b can be omitted, and

$$v = k_{\prime} \sqrt{x}$$

which is the ordinary equation as given in text-books; making x. small, the value $a x^2$ may be omitted, and then

$$v = k_{\prime\prime} x^{\frac{3}{2}},$$

a result which accords with the facts as well as with the theory, because it is clear that very small bodies must remain suspended in the fluid ($v = 0$), hence the exponent must be greater than one. Were the old formula correct, a body whose diameter was dx would have a finite velocity.

The above equation appears to hold for diameters as small as $x =$ 0.0001 mm. Dr. Barus (*U. S. Geolog. Survey Bulletin* 39) assumes Sp. Gr. quartz, clay, etc., 2.50, and from rate of observed subsidence computes for

$$\begin{array}{ll} x = 0.0001 \text{ mm. } t.\ 15^\circ\ C., & v = 0.0000278 \text{ mm.} \\ \qquad\qquad\qquad\quad t.\ 100^\circ\ C., & v = 0.000556 \quad \text{“} \end{array}$$

The above formula does not consider temperature, and gives $v =$ 0.000139 m m., a result fairly in accord with that of Dr. Barus.

The following table shows the value obtained from a discussion of eight of the thirteen minerals given in the table quoted:

Minerals.	Sp. Gr.	k.	a.	b.	(a. + b.)
Anthracite	1.473	25.36	.7815	.6267	1.4082
Quartz	2.640	100.4	.9030	.5195	1.4125
Pyrrhotite	4.508	140.1	.5248	.9159	1.4407
Chalcocite	5.334	140.5	.6396	.7902	1.4298
Antimony	6.706	191.4	.8799	.5485	1.4284
Wolframite	6.937	205.5	.9034	.4887	1.3921
Galena	7.586	283	1.0770	.3220	1.3990
Copper	8.479	187.6	1.0630	.3510	1.4140

The mean value of $(a + b)$ is 1.4156, which is nearly the same as $\sqrt{2}$, $= 1.4142$. This close coincidence of value as well as the more important fact of $(a + b) =$ constant, has a significance that the writer is unable to grasp, and should lead to renewed experiments upon spheres whereby the weight would be known and the influence of form would be constant.

The relation $\frac{v}{v_{\prime}}$ is not a constant for any two minerals, unless a and b are the same, for instance making x large and small in the case of galena and quartz.

$$x \text{ large, } \frac{v}{v_{\prime}} \frac{\text{galena}}{\text{quartz}} = 2.581 \text{ lowest value of ratio.}$$

$$x \text{ small, } \frac{v}{v_{\prime}} \frac{\text{galena}}{\text{quartz}} = 3.58 \text{ highest value of ratio.}$$

Having shown that the law of velocities is greatly changed for small values of x, it is suggested that it is probable that a similar modification will be found of the law governing the settlement of fine particles upon inclined planes (Vanners, canvas, etc.), and as perhaps more than 80 per cent. of all the ore stamped is under $\frac{1}{4}$ mm. diameter, it seems to the writer that there is an excellent field here for original investigation by the various mining schools, of the laws governing the separation of small bodies under $\frac{1}{4}$ mm.

Editors reprinting articles from this journal are requested to credit both the JOURNAL and the Society before which such articles were read.

ASSOCIATION OF ENGINEERING SOCIETIES.

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WATER SUPPLY AND SEWERAGE AS AFFECTED BY THE LOWER VEGETABLE ORGANISMS.

BY THE LATE CLARENCE O. AREY, C.E., M.D.

[Read before the Civil Engineers' Club of Cleveland, June 9, 1896.*]

IN taking up this subject, regarding the effect of lower vegetation upon our water and sewerage, it will first be necessary to study the nature and life-work of these minute organisms which are found everywhere, and to establish their place in the circle of the varied forms of life.

Man lives either upon other animals or upon vegetables. These other animals that furnish food for man live either upon vegetables or upon herbiverous animals dependent upon vegetable life. All animal life is therefore dependent upon vegetation for its existence. Upon what, then, does the vegetable life with which we see ourselves surrounded depend? It depends upon the gases in the air and in the soil in which it is developed. Water is part of the food of all life and it is not necessary to consider it in differentiating the various forms.

What furnishes the constant supply of the elementary gases upon which the higher forms of vegetable life depend? The life-work of the lowest forms of vegetation is to supply these gases. The bacteria, the yeasts, and the moulds do this work. They take dead organic matter as their food and reduce it to its original elements, which are mostly gases. Without them, all dead matter, unless destroyed by fire or cauterizing chemicals, would remain forever in the exact condition that it was in when death took place. When the Egyptians mummified their dead, they simply destroyed all of these organisms of decomposition. We are

* Manuscript received September 3, 1896.—*Secretary, Ass'n of Eng. Socs.*

all familiar with the fact that plants will not grow on fresh manure. This is simply because the bacteria have not yet reduced it to the elements necessary to feed the plant. It is also probable that the heat, produced by the chemical changes instigated by the bacteria, acts deleteriously upon the plant. The pea and the bean contain a considerable percentage of nitrogen, and upon investigating the roots of these plants we find that they are covered with a species of bacteria whose function it is to produce nitrogen. Clover is similar, and farmers have planted their fields with clover in order to render the soil more rich, that is, to replace the nitrogen that had been exhausted from the ground. Lately the experiment of inoculating the soil with nitrogen-producing bacteria has been made.

Now, as to the structure of the bacteria, yeasts, and moulds. The bacteria are the lowest forms of vegetable life that we have. They consist of single cells, and their function as a class is to reduce dead matter to its original elements. They are not all engaged in this work, however. Some are parasitic and live upon other forms of life. We all of us have our skins covered with a variety called staphylococcus epidermatis albus, and discovered by Dr. Robb, formerly of Johns Hopkins University, but now of Cleveland. Of those that are parasitic in their nature, certain ones eliminate a poison which is deadly to the host, that is, to the person or animal upon which they happen to find an abiding-place. Right here comes the all-important point regarding bacteria ; namely, how they produce disease. All life requires food ; all life gives off excretions. All bacteria absorb food ; all bacteria excrete other matter. The comparatively few disease-producing bacteria excrete poisons more deadly quantitatively than any known chemical poisons. These poisons separated from the bacteria will produce the same disease as the bacteria themselves, but do so more quickly because the living bacteria require time to multiply until they are numerous enough to produce a poisonous quantity of their excretions before the symptoms of disease show themselves. These same poisons diluted sufficiently, as in drinking water, may after a time render the person drinking the water incapable of taking the disease they produce when given in poisonous quantities. The poison-producing bacteria are the ones that we wish to keep out of our water supply, out of our houses, and out of our sewers.

The yeasts are slightly larger organisms and contain a nucleus. They are generally gas producers.

The moulds are slightly higher up in the vegetable scale; they branch and have fruit.

The yeasts and moulds are perhaps antagonistic to the bacteria.

The greatest enemy of bacteria is sunlight. If we take two sterile

gelatine plates and inoculate both with the same species of virulent bacteria, and expose one for half an hour to direct sunlight, and do not expose the other, the result will be that the exposed plate will contain no growth whatever, while the one not exposed will have a luxurious growth of the inoculated bacteria upon its surface.

In taking up the subject of water supply, let us first consider a river town. Suppose that a town is located on and takes its water supply from a river and that ten miles up the river is a small town which discharges its sewerage into the river, the question which at once arises is, will the health of the lower town be good? The answer to this question will depend entirely upon the amount of sewerage discharged by the upper town in proportion to the distance between the towns and the size of the river. Let us leave out the question of chemical waste and consider only the effect of the disease-producing bacteria that are carried in sewerage.

The sewage, when small in quantity, is discharged into the river and is immediately diluted with the river water. It is tumbled over and exposed to the sun, and at every tumble thousands of bacteria are destroyed. The bacteria are filtered through the green slime growing in the rivers, going to meet their death in the filtration, till, at the end of three or four miles, the water, upon examination, is found to be pure enough for drinking purposes. But if the sewage is once allowed in the river there is no limit to its quantity, and it soon gets beyond the power of the combating agencies of nature. As to the length of time that bacteria will live in water when in contact with the combating agencies the data is not exact. In some experiments they have died out in a few days from the time of their introduction into the water, while in others they have persisted for several weeks. But, on the whole, it is safe to say, if all source of infection is cut off from a body of water, that it will entirely purify itself of disease organisms inside of a few months.

In the investigations of the Massachusetts Board of Health which were carried on a few years ago, it was found that if given quantities of sewage were discharged through open beds of gravel at regular intervals, as many bacteria were found in the filtrate at the first discharge as in the waste matter discharged into the gravel bed. After the gravel bed had been in use some time, however, with the proper intervals between discharges, it was found that the filtrate running away from the gravel beds was free from a harmful percentage of bacteria. Upon investigation this was explained in the following manner: When the sewerage was first discharged through the gravel, the gravel was clean and no forms of life that are at war with the bacteria were present. During the intervals, the food for the warring elements being present, their seed became planted there and grew, thriving upon the bacteria and other material furnished by the discharges.

Now this applies directly to the water supply of all large cities where filter beds are used. Take a freshly made filter bed and the water comes through impure. After a little time algoid vegetation—the green slime—begins to grow at the bottom of the water and on the sides of the filter beds. As this accumulates the bacteria are retarded in the meshes of this fine vegetation, which in some way destroys the bacteria. Finally the meshes become so fine that even the water does not percolate. Then the bed must be cleaned, but no new filter bed, nor freshly cleaned one, should be used to supply a city with drinking water.

In the city of Berlin, some years since, a portion of the city supplied with a certain filter bed became short of water. The level of the water was raised two feet in this bed, to give a greater pressure and force more water through the filter. The result was an immediate outbreak of typhoid fever in the part of the city supplied by this particular filter bed, and in no other part of the city. The most plausible explanation is, of course, that these bacteria had been accumulated in large quantities in the meshes of the vegetation growing on the surface of the filter, and that the sudden heavy pressure had forced them through the filter before there had been time for them to meet their death at the hands of their natural enemies.

The new water-works of Berlin furnish us a lesson in the scientific way in which the question is handled. Beside the usual corps of engineers, they have two bacteriological laboratories located at two different points of supply. Dr. Proskauer, who has charge of these laboratories, was the first one to show that it was not the sand alone, but the algoid vegetation on its surface that formed the filter arresting the bacteria and in some way absorbing or removing the dissolved organic matter. After the beds have been cleaned the filtrate for the first forty-eight hours is rejected. There are in this system twenty-two filter beds, of which only sixteen are in use at one time, while the remainder are being cleansed. To avoid all possible cause of error from any flaw in the filter bed, the filtrate is examined daily in the bacteriological laboratories. Koch's three rules regarding filtration are rigidly enforced. These are:

(1) That the rate of filtration shall never exceed 100 mm. per hour.

(2) That the filtrate of each basin shall be examined daily while in use.

(3) That the filtered water containing more than 100 bacteria to the c.cm. shall be rejected or pumped back into the unfiltered reservoir.

The average number in the water as now supplied to Berlin seldom amounts to 50 bacteria to the c.cm. The unfiltered Tegel water averages about 200, while the former source of supply in Spree contained

from 10,000 to 100,000 or more to the c.cm. A filter conducted on these principles should reduce the bacteria in a water that is badly contaminated, in the ratio of 1,000 to 1.

So far sewage has only been considered in the way it may affect our water supply. Now, consider it by itself. In what forms do we find it, and how may the disease-producing elements which it may contain reach us? We find sewage in leeching cesspools, in earth-closets, in tight cesspools, and in the city sewers. The leeching cesspool stands in exactly the same relation to surrounding wells that the sewerage discharged into a river does to the purity of the river. If we have a small enough supply of sewage and a great enough distance, we are safe. The earth-closet, when supplied with the proper kind of earth, is a sanitary appliance. The proper kind of earth is a dry loam, or surface garden soil dried without heat, which contains the forms of life that combat the noxious bacteria.

Before taking up the other forms of sewerage disposal, let us discuss the manner in which this sewerage can be harmful to ourselves. How do the disease-producing bacteria, that may be contained in the sewage, reach us? Are thev carried through the air if the sewage is exposed? This is impossible unless the sewage is first dried, desiccated, and then exposed to the winds. As long as it is moist it is harmless to the air we breathe. Tests of the air in some large city sewers show a greater purity than the average well ventilated schoolroom when full of students. These organisms can only be carried to us by contact. The organisms that make the odors are not the disease-producing ones. The odoriferous bacteria are probably intended to keep us away from the foul matter in which they live. They are danger signals. They say "Don't touch." That great bug-bear—sewer-gas—is a harmless old fellow. He never hurt anybody any more than any other gas might do by reducing the amount of oxygen. It would be difficult to get a leak of sewer gas into a room from a well ventilated system of plumbing, that would vitiate the atmosphere anything like the amount that it is vitiated by an ordinary gas burner which uses, during a given period, at the least as much oxygen as eight persons. The elaborate system of back ventilation which is required of the plumbers by the health laws of most of our cities is a good thing to keep the plumbers busy, but all that it accomplishes otherwise is to keep an occasional whiff of air in a ventilated waste pipe from entering an apartment at the moment that a trap is siphoned.

Dr. A. C. Abbott, of the Laboratory of Hygiene of the University of Pennsylvania, during the winter of 1894–95 conducted some experiments upon animals, as to the nature of sewer gas and of the gases arising from decomposing organic material. He took some rats,

rabbits, and other animals, and placed them in glass jars. Over some he passed a continuous stream of sewer gas; over others, gases from decomposing material. This was continued without interruption for five or six weeks, and none of the animals suffered a loss of appetite, nor seemed otherwise any the worse for wear. And these are the animals which are especially subject to disease in laboratory experiment. To make the reason for this clearer, the statement that the disease-forming organisms are not gas producers may need a little explanation. The bacteria of decomposition almost universally produce gases. The parasitic bacteria seldom produce gas. They live on our bodies and in the passages and chambers of our bodies that communicate with the external air. We have a variety which lives in our intestines, that is a gas producer and which, if introduced into other portions of the body where bacteria do not normally belong, may produce disease. But, of the parasitic bacteria, a few are normally disease producers, and of these normal disease producers I do not know of one that produces gas. Even if they did the gas would not of necessity be poisonous.

What is wanted of a sewerage system is to take away the noxious material as rapidly as possible.

Returning for a moment to the different forms of sewage disposal, we find the tight cesspool harmless because the sewage in it is moist. It gives off plenty of gas however, which is inhaled by every visitor to the apartment above. City sewers, if well ventilated, tight, and well graded, are all that can be desired—but where are sewers to empty? This is the most difficult question to solve in this direction at present. A city like Buffalo, which has a river at its doors with a current of eight miles an hour, into which it can empty its sewage, and Lake Erie from which to draw its water supply, can easily solve the problem. A small town can use the irrigation and sewage-farm method. Where land is cheap the farm products will nearly pay the expense of running. This system depends for its success on the bacteria of decomposition, and other vegetation. The experiments of the Massachusetts Board of Health, already referred to, explain the principles of this system. The works at Freehold, N. J., are a good illustration of this type. The city sewers discharge into a large tank or basin. When this basin is full the contained sewage is allowed to flow with full head through one of the distributing systems. There are, if I remember rightly, six different fields used in alternation. The discharged sewage, before flowing onto the fields, first flows through a barrier of broken stone, where the algoid vegetation does its destructive work. The alternation of the fields gives this vegetation time to develop—being swamped with food—and the several days' exposure to the sunlight between the floodings destroys the noxious bacteria that may have escaped the barriers of broken stone.

The solution of the problem in this city has been in the past merely a stern chase, moving the source of supply farther out into the lake, while the growth of the city is forcing larger supplies of sewage farther out into the lake. Taking the system just as it is, however, a daily bacteriological examination of the water at the intake would show whether the water was fit for drinking purposes, and the Water Works Department could keep the people of this city informed on this point.

Before closing, I should like to say a word about house filters. The old charcoal, or a sand filter, might do if they were only used one day in four or five, and exposed to air and light during the intermediate days. The wire filters that screw on the faucet and reverse are an abomination ; the retained bacteria have time to multiply, and are then slowly set free when the filter is reversed, and, after a time, the water passing through in either direction will contain more bacteria than is contained in the same amount of water in the pipe supplying the filter. I have noticed a pressure filter which contains a film of paper pulp to be removed daily. I should think that these might be very effective, although I have never seen a report of the condition of the filtrate from such a filter. Then we come to the earthenware filters. These answer their purpose if—and the if is a large one—if they are sterilized by heat about once in four days. It has been shown by experiment that after this period there is more bacteria contained in the filtrate than in the water that enters the filter. The explanation is that it takes about four days for the bacteria to grow through the pores of the earthenware, and after this is accomplished these earthenware walls become a breeding-place. If, however, one removes the earthenware portion and either lays it on the coals till raised to a red heat, or puts it in a steam chamber for half an hour, and repeats this operation every fourth day, the filter will then be perfectly safe. It is much better, however, to have the city furnish water that is perfectly safe for everyone to drink.

THE TESTING OF COALS.

By Arthur Winslow, Member of the Engineers' Club of St. Louis.

[Read before the Club, March 18, 1896.*]

The importance of a determination of the properties and relative values of different coals is so patent that no argument or illustration is needed in support. The problem has been under investigation for many years and has been attacked in various ways. There has been a development in the methods employed, and a clearer understanding has been reached of the results desired and of what it is practicable to attain. But much remains yet to be done.

With the long continued and abundant use which some coals have enjoyed, their individual values and adaptabilities for certain purposes have been pretty well established in practice. There are many other coals, however, concerning which this is not the case. Further, with most coals the questions of relative values for all uses and under all conditions are in a very unsettled state. The conditions of actual use are not sufficiently uniform, nor have the results been recorded with the exactness necessary for close comparisons. There is need of supplementary tests. It is true that there are few coals of which analyses have not been made. Calorimeter and boiler tests of many have been recorded. But, unfortunately, the results are generally not comparable or are incomplete. Some of the samples tested were of picked specimens; some represented sections of the coal bed; some were averages of car-load or other lots; some have been tested fresh from the mine; others after long exposure. The apparatus and methods used in different tests have been different, and not always those best adapted to the special coal. Some tests have been conducted by reliable men, others not. Therefore, a series of examinations and tests which will furnish information and results of uniform reliability and closely comparable, is still a desideratum; such a work will be of great value to all producers and consumers of coal.

This paper is the outcome of a plan of the writer's to conduct a study of North American coals along these lines. The work is already begun, but the plans and methods are not entirely matured. One object in presenting the matter at this stage is to lay these plans, so far as formulated, before you for discussion and suggestion. It was originally the writer's hope to be able to incorporate some results in illustration,

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but unavoidable delays in the prosecution of the work have prevented this, and have made the presentation perhaps a little premature. It is planned, however, to present some of the results in later communications.

The problem which first presents itself is the devising of a ready and practical method of determining the properties of coal with reference to actual uses. Such determination will establish directly the relative values of coals, and will reveal their special adaptabilities. The solving of this problem involves a consideration of the principal uses to which coals are put, and of the requirements of such uses. These may be included under the following five general heads, arranged about in the order of their importance:

(1) Steaming.
(2) Coke making.
(3) Domestic use.
(4) Gas making.
(5) Forge use or blacksmithing.

For steaming purposes the requirements vary according to the character of the furnace and boiler and the draught employed; they vary according to the use to which the boiler is put. In general, high evaporative power is desired in the coal, but in some cases quick steaming qualities are of more importance than heating value. High ash and moisture percentages, and the presence of harmful impurities, such as iron and sulphur compounds, are always objectionable, as is also a coal which makes a hard clinker, or much clinker or cinder of any kind. A too fusible and strongly caking coal cannot be regarded as best for steaming; yet, on the other hand, one which will not fuse or cinder at all, and is at the same time soft and friable, is liable to be wasted through the grate bars and up the stack. A good steaming coal should not make much soot, as this clogs the pipes and reduces the amount of steam made. It should reach consumers in a good condition, and to stand storage it must not slack much in handling or hauling, or by exposure to the weather. The results of test and practice lead to the conclusion that, up to a certain point, the coals high in fixed carbon have the greatest evaporative powers in the common types of furnaces and boilers; this quality, combined with the readiness of ignition and the free and complete burning of semi-anthracite and semi-bituminous coals, make them the steaming coals par excellence. They also produce little smoke and soot, which are important considerations in many uses.

The essential of a coking coal is that it coke. This some coals will not do. The second important consideration is the quality of the coke. Very inferior cokes find sale for domestic and some other uses, but they command a low price and their sale is limited. They can generally only

be made on a slack basis. Cokes which are of first grade for some uses occupy a second place for others. For iron smelting a very strong coke is necessary, one of moderate ash and very low in sulphur and phosphorus; for foundry use a coke of similar composition is required, but equal strength is not necessary. For lead smelting, and for treatment of ores of the precious metals, neither so strong nor so pure a coke is essential; the percentages of sulphur and phosphorus do not cut such a figure here; a moderate amount of ash is, of course, desirable, but this is qualified by the composition of the latter. In the manufacture of water-gas low sulphur is important in a coke. The density of a coke and the porosity are physical characteristics of some importance in metallurgical use. The amount of coal required to make a ton of coke, and the length of burning necessary, enter into the question. The lustre and fracture of the product affect its salability in the market.

For domestic use various kinds of coals are used, according as to whether they are to be burned in furnaces, ranges, self-heating base-burners or open grates. Dirty, impure and freely slacking coals are always objectionable, but small amounts of impurities are not of the importance here that they are with coals for metallurgical use. For large house furnaces good anthracites are undoubtedly the best. For ranges and base-burners anthracites are also good, but dry, semi-anthracites are sometimes preferred on account of their greater ease of ignition; a smoky, caking or intumescing bituminous coal is not adapted to such uses. In open grates almost all good coals are burned, the choice being very largely a matter of taste. A clinkering coal or one high in sulphur is bad in domestic as in steaming uses, but a coal which makes a clinker with the temperature of a boiler furnace may not do so with the lower temperature of domestic fires.

For gas-making the essentials are a large quantity of volatile hydrocarbons of good illuminating power. Sulphur in combination with iron, as iron pyrite, is the principal harmful impurity, and it has to be removed with lime or sponge. A large amount of ash is injurious in that it reduces the yield of gas per ton of coal and impairs the quality of the coke. The coking qualities of the coal also affect its value for gas use.

A forge or blacksmithing coal should be a moderately caking coal, containing very little sulphur and low in ash. Too fusible and pasty a coal is objectionable.

To find the value of a coal for these uses requires, therefore, the determination of certain facts of composition, of physical character, and of behavior in burning. To satisfy these demands the following scheme of work has been planned:

(1) An inspection of the coal at the mine.

(2) A personal collection of samples at the mines.

(3) A proximate analysis.

(4) A fuel test by calorimeter.

(5) A laboratory test of the coking and gas-producing qualities.

(6) A study of the best methods of burning, of the steaming value, of the durability in transportation and storage, and of the special adaptabilities as revealed in actual use.

Ultimate analyses, boiler tests, coke-oven tests, gas analyses, etc., will be made in addition only in special cases.

The experimental part of this work consists essentially of sample tests; but these are supplemented by observations upon and inquiry into the behavior in actual use. At the same time results of commercial tests heretofore made, and all other data obtainable, are industriously gathered. It would, of course, be preferable, and the scheme would be ideally perfect, could working tests be made for all the different uses of coal; but with a private undertaking it is manifestly impracticable to conduct tests of a great number of coals on such a scale and in such variety as to reproduce all conditions of practice, and it is recognizedly impossible to attain these conditions with tests of small samples. Hence, this compromise or combination of direct experiment and of observations on practice was decided upon as the most feasible plan of work, the results in the one line acting as guides and as supplements to those in the other.

Considering the plan of work in detail, the inspection of the coal at the mine and the personal collecting of the sample are considered of first importance. They not only enable one to vouch for the character of the sample, but they permit the necessary observations for securing a proper sample. At the same time valuable information can be gathered at the mine bearing upon the uses and behavior of the coal. The sample desired is one which will fairly represent the product of the coal bed at the special locality, as it can be shipped after proper preparation for the market. The coal shipped from any bed may vary according to local conditions and according to the care exercised at the mine in preparing it for market. The variations of the bed are ascertained by underground inspection. A sample is then selected which will represent a fair average of the product. This is generally done by collecting a large number of lumps from market or mine cars and from different benches of coal, and coming from different parts of the mine—between ten and twelve bushels in all. These are then broken down, well mixed, and successively quartered and broken until a sample consisting of about half a peck of small coal is left. In addition, about a peck of lumps of egg size are selected at random for special tests of hardness, burning, etc. These are shipped to headquarters at once and there the small sample is transferred to an

air-tight glass jar for future use and preservation. Such a sample is a fair average and is considered better, for other reasons, than one consisting of chippings from across the face of the bed in the mine.

The proximate analysis is the ordinary one, covering the determination of the fixed carbon, volatile hydrocarbons, moisture, ash and sulphur.

The fuel value test will be made in some form of oxygen calorimeter, which has recently received such strong endorsement for the determination of the qualities of coals by the committee on a "Standard Method of Conducting Locomotive Tests" of the American Society of Mechanical Engineers. The apparatus will probably be that devised by Mr. George H. Barrus, and described by him in Vol. XIV of the *Transactions of the American Society of Mechanical Engineers.* It is very simple in operation, and is free from the objectionable chemical reactions of the Thompson calorimeter. It appears to be capable of yielding amply accurate results for the purposes in view, and is cheaper and less complicated than the bomb calorimeters of Mahler and others. As stated, boiler tests are not contemplated excepting in special cases. Such are always as much a test of the boiler and furnace as of the coal, and it is not practicable to extend the tests of a coal to all the different types of boilers. Because a certain coal with one furnace yields a high or low result as compared with another coal, it does not follow that the same result will be obtained when different furnaces and boilers are used. Hence the usefulness of such tests in establishing the relative value of coals is not so great as might be imagined, and may even be misleading.

The tests of coking properties will be made in small fire clay crucibles. The results thus obtained may not be final in fixing the quality of the coke, but they will certainly establish the fact of coking or non-coking properties, and it is believed that careful observations and comparisons with results obtained from similar tests of standard coking coals will lead to much more.

The gas-making qualities will be inferred from the results of the analysis and from the behavior during the coking tests. Actual tests of the candle-power of the gas cannot ordinarily be attempted.

Tests of hardness, of igniting qualities, of fusibility or clinkering of ash, and other minor points, will be conducted so far as practicable, and these will all be checked by notes on and observations of the results reached in practice.

By these means it is believed that a series of facts and notes will be obtained, entirely comparable and subject to the same personal equation, which will be sufficient to establish the essential qualities of the different coals. These are, of course, always liable to local and temporary variations, according to the care of the miner, etc.; but such variations

are generally exhibited in the amounts of ash and sulphur, and in the proportions of slack and lump coal, which can be readily determined by simple inspection and proximate analysis, and can be corrected to the standard.

The results, though expressed in absolute terms, do not, of course, give absolute values. These are always relative to time and place. Without knowledge of these factors the money value of a coal can never be estimated. Many a hasty verdict has been passed and an enterprise obstructed by ignoring this fact. A coal which may be of very poor composition and of inferior qualities, to the extent that in some localities it would be practically worthless, may elsewhere, under different surroundings and market conditions, be a very merchantable product; and the reverse is equally true. This consideration, though somewhat aside from the subject of the paper, has a bearing upon the use of the results which we hope to attain.

METHODS AND RESULTS OF STADIA SURVEYING.

By F. B. Maltby, Member of the Engineers' Club of St. Louis.

[Read before the Club, June 3, 1896.*]

The theoretical and mathematical discussions of the method of measuring distances by the use of the stadia have been quite full, both in text-books and papers and discussions before this and other engineering societies. Without in any way wishing to cast disparagement on these theoretical discussions, the author has thought that some notes on the practical use of the stadia as gained principally from his own practice covering a number of years may be of interest.

Your attention is first called to methods employed, afterward to some examples of results attained. First to be considered under the head of methods are the appliances used. The old adage to the effect that "any one can do good work with good tools," but that it takes a mechanic to produce good work with poor tools, may be true of some kinds of work, but the author does not think that even a mechanic or a trained observer can do first-class topographical work with poor instruments.

The first of the appliances is the transit or theodolite, and I wish to urge very strongly, at the start, the importance and necessity of a first-class instrument. By this I mean not only one of good mechanical workmanship, but one designed and adapted to this work. In the opinion of the writer, one of the principal reasons for the feeling of distrust, held by some engineers, of work done by the stadia method is derived from the unsatisfactory results obtained by the use of instruments unsuited to the purpose.

An instrument should possess the following qualities: It should be thoroughly rigid, and heavy enough to provide stability in a strong wind. The graduations should be accurate, deeply cut and clear, and the horizontal circle marked to read from left to right, from 0° to 360°, and should read by vernier to 20″ at most. The telescope (the important part for long range and accurate reading) should be powerful, magnifying not less than 30 diameters, and the field should be perfectly flat and as large as possible. To meet these conditions means a very much larger object-glass and longer barrel than ordinarily used on engineers' transits. To save light absorbed by an extra lense it should preferably have an inverting eye-piece. The instrument should have a vertical circle divided and reading by vernier to the same divisions as

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those on the horizontal circle, to avoid confusion in reading. The vernier should swing from the horizontal axis of the telescope and be provided with a tangent screw for moving it independently of the rest of the instrument. It should be provided with a level tube so adjusted that when the line of collimation is horizontal, and the vertical circle reads zero, the bubble will stand in the center. This arrangement will enable one to read vertical angles accurately, regardless of the fact that the plates of the instrument may not be exactly horizontal, and it is strictly essential where elevations are to be determined with a reasonable degree of accuracy with vertical angles.

The highest grades of instruments ordinarily made in this country for railroad and municipal work do not meet all the above requirements. In consequence, where the work in hand is extensive enough to warrant the expense, it is desirable to have an instrument made for the purpose.

For use on the topographical survey of the City of St. Louis, Mr. B. H. Colby had two made, one by Fauth and one by Buff & Berger. The writer, for use on work of such an extent as to prohibit the cost of an instrument made to order, has had a vernier level attached to a high-grade Buff & Berger transit, and it has given excellent satisfaction, the only fault being that the telescope is hardly strong enough. The Mississippi River Commission have recently had made two theodolites especially designed for this work. By courtesy of Mr. J. A. Ockerson, under whose direction they were designed and made, I am able to give the specifications of this instrument. In the writer's opinion this is the model topographical instrument.

The matter of rods and their marking has been much discussed. Only recently this club listened to a paper on the subject. It is found, all things considered, that a clear, well seasoned white pine board, 5 inches wide, 12 to 14 feet long, and about $\frac{5}{8}$ inch thick, gives the best satisfaction. A jointed or hinged rod has been advocated, but a joint that is rigid and will stand the rough usage to which a rod is put must be very cumbersome and heavy, and, in the writer's opinion, the small matter of slightly more portability is hardly worth what it costs. The rod should be lightly shod with strap iron at each end to prevent splitting and wearing off of corners. A small hole drilled in the center of the shoe, and the use of a headless nail driven in the top of the stake will enable a rodman to always hold the rod on the exact point marking the station. It is also quite a help to a rodman in holding his rod plumb in a high wind—a small detail but a convenient one. The rod should be given two or three heavy coats of white paint. The figures should be black, with the larger units painted red, for ease in "counting up." The marking should be symmetrical with the center of the board, in order that either end may be held up.

The shape of the figure to be used is largely a matter of individual preference. I suppose that nearly every one who has used the stadia extensively has designed, tried and usually discarded at least one figure. In looking over the available literature on this subject I have been somewhat amused to note the great variety of figures and ways of mark-

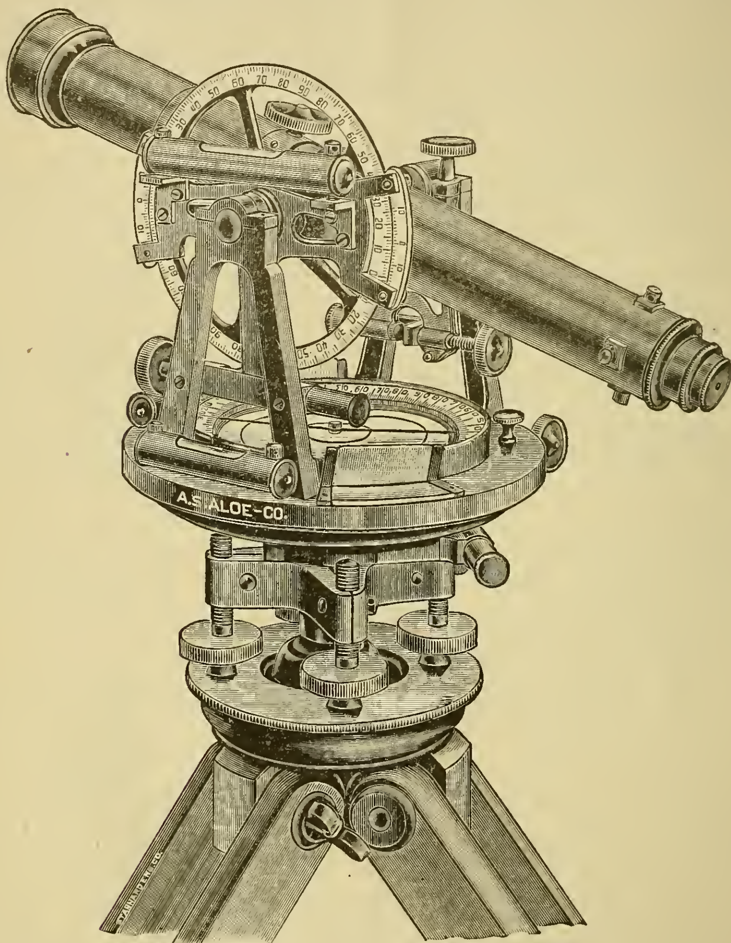


Fig. 1.

Topographical Theodolite made for Mississippi River Commission, 1895.

ing a rod that have been advanced. Many of them have many meritorious features under certain conditions. It is, however, desirable to keep the colors together, as much as possible, by using large figures which will largely prevent their running together when the air is unsteady. It is better to use large figures and subdivide them by means

of angles and points, than to use a multiplicity of figures. The writer has never used a figure that as fully answers all the requirements as that adopted on the survey of the Great Lakes, and commonly known as the "Lake survey figure," and universally used on the topographical surveys made by the Mississippi and Missouri River Commissions, and also on the topographical surveys of St. Louis and Baltimore.

The general practice of marking rods on work with which the writer has been connected has been to measure a base of such length as is estimated will be the average distance between stadia stakes, measure the interval subtended a number of times, take the average and subdivide the rod proportionally. Or, better yet, measure a number of distances covering a range likely to be used in practice, determine the interval for each distance, take the mean of all and subdivide the rod proportionally. This method has one advantage, and that one is of considerable importance, that is, distances are read directly from the rod and can be platted as read without further reduction being required. Where the work is to be platted on a scale of 1 in 5,000, or smaller, and where, as in contour work, the vast majority of the side shots are for elevation only, and their exact position not essential, and as the rods can readily be graduated to read with an error not to exceed 1 in 600 to 800 or closer, the error of location is well within the limits of ordinary platting. As the side shots comprise 90 per cent. of the readings taken, the advantage of the ability to plat them without any reduction, and on work of wide extent, is readily appreciated. Of course it is understood that the rod reading is correct only for the distance for which it was graduated. Another disadvantage is that the interval is not always determined under the same conditions actually met with in use of the rods.

For work on a large scale, say 100 to 400 feet per inch, and where a large amount of accurate detail is to be determined, it is possible that the method advocated by Mr. J. L. Van Ornum in his paper on the Topography of the Survey of the U. S. Mexican Boundary, in *Trans. Am. Soc. C. E.*, is the best.

This method is to subdivide the rods into standard units, uch as feet, yards, meters, etc., and to determine the rod interval and use a conversion table for reducing rod readings to distances. It has the advantage, among others mentioned by Mr. Van Ornum, of being able to keep standard rods in stock and ready for use, and that the interval can be determined under all conditions of practice. Instead of a small number of readings over a picked base, the interval can be determined by running between known points and observing under all the conditions of sunshine and shade, and all kinds of temperature and state of humidity; in other words, a constant determination of the interval whenever prac-

ticable. And in case it changes, as it does, it is only necessary to change or correct the tables instead of repainting the rods. The disadvantage, as before stated, lies in the necessity for converting all rod readings to distances, a process taking some time.

The wire interval to be used is quite important. The smaller the angle subtended by the cross wires the longer will be the distance at which a rod of given length can be read, and the wider the angle the greater the degree of precision with which the divisions on the rod can be read and subdivided. The choice of the angle depends somewhat on the scale to be used and the amount of detail and accuracy required. The choice of a large or a small angle is also practically limited to a rather small range, and it should depend on and be consistent with the power of the instrument. If the angle is too small, the figures on the rod will be so small that they cannot be distinguished at a long distance, or if a rod divided into standard units is used, the error in reading the interval affects the determination of the distance to a greater proportion. Thus if the interval is 1 in 100 then an error in reading the interval .01 foot makes an error in the determination of the distance of one foot. If the interval be .5 to 100, or 1 in 200, then the same error in reading the interval makes an error of 2 feet in the determination of the distance. If the angle is too large, there is trouble in taking in both wires at the same glance, and second, unless the telescope has a perfectly flat field, it is difficult to bring all the wires into the same focus.

In the ordinary American transit the angle subtended by the cross wires is about 34 minutes, or the space subtended on the rod is $\frac{1}{100}$ of the horizontal distance from the telescope. With the instruments in use on the surveys of the Mississippi River the most satisfactory interval has been found to be about .8 foot per 100 feet. On the Missouri River .8 to 1.0 per 100 are used; on the Baltimore survey 1 foot and 1.5 per 100 foot were used; and on the St. Louis topographical survey about .9 foot per 100. In this connection the standard unit for horizontal distance to be used, whether feet, yards or meters, is important as affecting the size of the spots, as it is quite essential that for speed and accuracy in counting up, the rod should be decimally divided, whatever the interval or whatever the unit of distance used. Now if an interval of 1 in 100 is used and horizontal distances are to be measured in feet, then to carry out the decimal system of subdividing the rod 10 feet on the ground is represented by 0.1 foot on the the rod, making a spot too small to be distinguished or subdivided except at short distances. On the U. S. Coast Survey, the survey of the Mississippi River, in St. Louis, Baltimore and other places, the meter is the unit of distances. Without entering into a discussion on the merits of the metric system, I will only say in this connection that one advantage of its use is that it is found

that the 10-meter spots are of the best size for the usual range of work. On work of a private nature, or for corporation, the yard possesses the same advantage as the meter in this respect, with the additional advantage of being more readily converted to the usual standard unit of feet.

Coming now to the field work, we have first to consider the organization of the field party. It should consist of an observer, a recorder (who should be a rapid writer, and one able to make neat and legible figures, and, preferably, one able to assist in reducing the notes), and such a number of rodmen as will keep the observer fully occupied. It is economy to have enough rodmen so that if there is any waiting to be done the low-priced men may wait on the observer rather than to have the high-priced observer waiting for the rodmen to get around. On work of considerable detail and large scale, where the rodmen have only short distances to walk between shots, two may be enough, while in open rolling country with little detail and where a small scale is used, four may be required. Where there is timber or brush there should be the necessary number of axmen.

A certain piece of work requires that a certain number of points be located; obviously the progress of the work depends on the speed with which the observer is able to locate these points. Now a good observer in open country and with the detail usually required on scales, say of 500 feet to an inch, or larger, can locate 500 points per day. There are men who can do more. Counting the time in getting to and from the field and the time lost in going from one stake to another, there will usually not be more than five and a half to six hours per day for actual observing, or say one and a half shots per minute, or forty seconds per shot, and as each shot means, first of all, directing the rodmen more or less, pointing the instrument, reading the distance, the azimuth and the vertical angle, it is self-evident that a man cannot do his own recording at this speed, and, in fact, it takes a rapid recorder to keep up with the observer. To attain this speed there must always be a rod up ready to be read, and as it only takes a wait of about twelve seconds between each shot to reduce the amount of work done 25 per cent., the economy of having all the rodmen required is apparent. I enlarge on this subject because there seems to be an impression that where only a limited amount of work is to be done, an observer who can record for himself and one or two rodmen are all that are necessary. Topographical work can be done in that way, but not economically.

The success in obtaining the required results, which are the objects in view in making the survey, depends almost entirely on the discretion and experience of the observer. It has been said that successful topographers are born, not made. This I believe to be true only in a limited sense, as I believe anyone with intelligence sufficient to grasp

the idea of what is required can become a topographer. Generally speaking, a topographical survey, for whatever purpose required, is made, as the name indicates, with the object of showing on paper the configuration of the surface of the ground covered. It may also, and if made on a large scale usually does, show artificial features and limits of culture. The successful observer will keep the former idea in mind and locate such points on the ground as are the controlling points in determining the features to be reproduced on the map. Shots taken indiscriminately not only take time in locating points not required, but may be misleading, inasmuch as they may not be controlling or limiting points.

The scale to be used on the map must also be borne in mind. It would manifestly be absurd for one to locate with the same detail features which are to be platted on a scale of 1 in 10,000 and 5-foot contour interval as those to be platted on a scale of 2 or 400 feet to the inch and contour intervals of 1 or 2 feet. Another desirable requisite is to cover the entire area to be surveyed with the same detail. These matters of the distribution of located points and the scale used are important. I have seen topographers, and not always new ones, who would locate points over a limited area with such detail that it was almost impossible to get them all on the paper, they were so close together, and then there may have been a space of a thousand feet or more in which no points were located.

Another matter in the distribution of located points to be observed, especially in gently rolling country, is that (if you will permit the expression) of running across the contours, that is, when the view from the instrument is obstructed by timber, weeds, brush, etc., let the lines to be cleared out be in a direction at right angles to the contours, or in open country let the paths gone over by the rodman in giving shots cut the contours at right angles. This may seem a small matter in detail, but on such small matters the successful map may depend, and the knowledge of the non-observance of this point by some topographers leads me to speak of it.

In the matter of sketching in the field, my experience leads me to take an entirely opposite view to that advanced by Mr. B. H. Colby in his very able paper on the topographical survey of St. Louis, presented before this club some time ago. Mr. Colby's idea is that sketching is entirely useless, and worse than useless, as it is a waste of time. He advances the idea that by the adoption of a proper nomenclature in keeping notes the controlling points may be positively identified and any sketch is useless. To this I must take exception. I cannot conceive of any system of notes being as plain to one entirely unacquainted with the ground surveyed as a well executed sketch. It has been said

that "a sketch is a universal language that is understood by any one, and requires no key or guide to its meaning." It is essential where others than those who did the field-work do the platting that the notes, or combination of notes and sketches, should be entirely clear and subject to but one interpretation. I do not advocate the indiscriminate sketching without scale in the pages of the note-books, as the value of the sketch then largely depends on the artistic ability of the observer to reproduce on the sketch his impression of the country before him, or the way it looks to him from his point of view. The result as obtained from the platted notes may be entirely different. A description of the system of sketching in use in the survey of the Mississippi and Missouri Rivers will embody my idea of what a sketch should be. This system was devised in the office of the Mississippi River Commission, and has

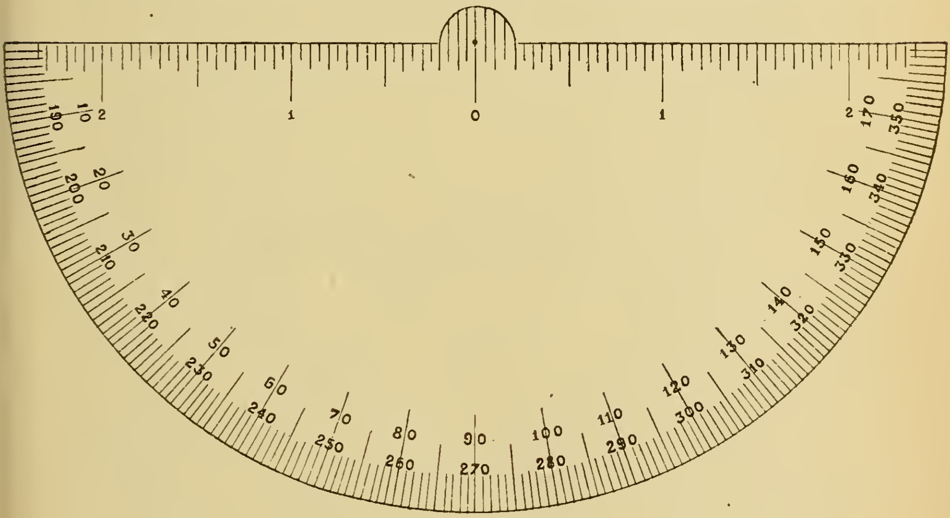


FIG. 2.

been adopted for their work and that done by the Missouri River Commission.

Sheets of paper are provided, 8 inches by 10 inches, ruled with light blue ink into squares, the lineal dimensions of which are some even division of the scale employed on the maps. A semi-circular celluloid or composition protractor (shown in Fig. 2) is used. It is 5 inches in diameter and is divided into degrees and numbered as shown. The edge along the diameter is divided into the scale to be used in platting (in this instance 10 parts to the inch, or a scale of 1000 feet to the inch) and numbered from the center each way. At the center there is a projection through which is a small hole at the exact center of the protractor. This hole is

also the zero of the scale. A pin fastens it to the paper and board. A very light drawing-board, about 15 inches by 20 inches, and a few thumb tacks complete the outfit. The method of using is as follows:

After having observed at the first station or stadia stake, the observer selects a point on the sheet as a starting-point to represent the station just observed from, and with the pin through the protractor at this point and using the ruled lines as north and south lines from which azimuth is platted, and using the scale on the edge of the protractor for distance plats the points located and also the next instrument station. These platted points are suitably joined and such information as is required is written on the sketch. At the next station the position already platted is used and the points located from it are treated in the same way, and thus the sketch grows as the work proceeds. When the edge of the sheet is reached another one is pinned down lapping over it a little, and the platting proceeds and may cover any number of sheets, the first being taken up from the board to make room for additional ones. The sheets are numbered and properly marked at their joining edges so that they may be readily placed together again in their proper position. Elevations are not platted and abrupt changes in elevation are indicated by hachures or approximate contour lines. No attempt is made to make a finished or artistic drawing, but the information required is placed on them in the quickest and clearest way possible. Not all the points located are platted, but only those sufficient for controlling points. Neither is it expected that the platting will be as accurately done as it will be done on the maps, but sufficiently so to show the relative position of located points and the proper way of joining them.

This method of sketching gives to the stadia method with transit the advantages claimed by the advocates of the use of the plane table, that is, the sketching or filling in is done by the observer on the ground, and with the features to be represented before him. It is without the disadvantage of the cumbersome board, tripod, allidade, etc., etc., of the plane table. There is a further advantage in this method that work can be done in any kind of weather that instrumental work of any kind should be done in, while with the plane table and large sheets, which are also sometimes used as the finished sheets, work cannot be done in damp weather or in a high wind.

Another advantage in sketching or platting the work as it proceeds is that it always shows to the observer what ground has been covered, and points out in what direction to proceed to cover the remaining ground to be surveyed. This is especially desirable in work of considerable extent and with an observer lacking in the quality known as "location."

The extra time required, if any, is very small; usually the observer

can do the platting and sketching while the recorder is going forward and setting up over the next stake. With this method nothing is left to the memory of the observer. On extended surveys such as those undertaken by the Government, or of areas of considerable extent for corporations, and where possibly the field party is subsisted in camp and the facilities for mapping are limited, all the energies of the party can be directed toward field work, with no delay caused by keeping notes platted and with the consciousness that everything surveyed is a matter of record, and will not be forgotten or confused. On completion of the field work, the remainder of the field party can be discharged and the observer can repair to an office where facilities may be had, and the

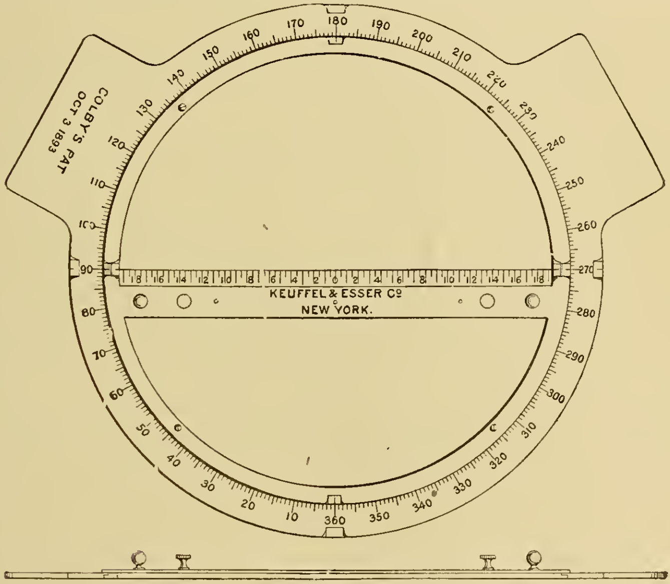


Fig. 3.
The Colby Slide-Rule.

map worked up at his convenience, or notes and sketches can be sent in and worked up equally well by one who has never been over the ground. Mr. Colby cites a railroad yard, with its complications of sidings, switches, cross-overs, etc., as being a test of the utility of his method. I can hardly imagine any system of notes that would make the different features of such a yard as clear to one who had never seen it as a well-executed sketch as I have described.

For reducing and platting the notes, suitable appliances are as essential as the field instruments. Differences of elevation being determined by vertical angles, it is necessary to reduce them to differences in feet

and tenths, or such units as may be used. For this purpose, numerous tables and diagrams have been devised. The most extensive and best arranged tables, so far as my own knowledge extends, are those prepared by Mr. J. A. Ockerson and Mr. Jared Teeple. They are quite complete, covering all angles and distances met with in ordinary practice. They are to be used where the distances are read in meters and differences in feet desired. Where other units are to be used, a reduction would be necessary.

So far as I have seen, the most successful device for obtaining these differences is the Colby slide-rule, a device described in Mr. Colby's paper, before referred to. With the use of this rule, horizontal distances may be read in feet, meters or yards, or any other unit, and the differences obtained in feet or the same unit as is used for horizontal measurements. As speed is one of the essentials of these reductions, I may say that I recently kept a record of the amount of work done with this rule by one man, in the office of the Missouri River Commission, and without his knowledge. During 54 working hours, he reduced 192 differences per hour, or a little over 3 per minute. Working under pressure, or for a record, one could exceed this rate considerably.

In platting the notes, the stakes are platted either by the use of rectangular co-ordinates or by polar co-ordinates, using a protractor printed on the sheet, and a parallel ruler for direction and a diagonal scale for distance. The advantages of the first method are that specially prepared protractor-sheets are not required. The co-ordinates are worked out and adjusted before platting, and any error in platting affects only one stake. Also in checking on the known position of well-located points, the error of closure is that of the field-work alone, and does not include errors of platting. The disadvantage lies in the time required.

The second method has a considerable advantage in the time required, and with suitable appliances can be made very accurate. The disadvantages are the necessity of specially prepared protractor-sheets, the fact that an error in platting one course affects all subsequent stakes depending on it for position, and also the disadvantage in closing on known points of deciding how much of the error is due to platting and how much to field-work.

During the past summer I had 295 circuits platted by polar co-ordinates for temporary use. Afterwards, the co-ordinates of the same stakes were computed. The average closure obtained by platting was 1 in 764. Average closure of same work by computation is 1 in 1228. These average figures do not show a true comparison, as in many cases the closure differed very widely and sometimes with the opposite sign.

For plating side-shots, some form of protractor is used. Two very good ones have been devised, and each has been used by the writer with

considerable satisfaction. One, by Mr. J. A. Ockerson, is made and sold by A. S. Aloe & Co., and fully described in their catalogue, and one by Mr. B. H. Colby, described in his paper before mentioned, and is made and sold by Keuffel & Esser Co.

The first is fastened to and swings on a pin-point through the platted position of the stake on the paper. Where the platting is done directly on what becomes the finished map, the objection of marring the paper in this way is quite a serious one. If the work is first platted on protractor-sheets, and then transferred to the detail-charts, this objection disappears. It has also the advantage of costing only about one-third as much as the other one.

The second one has the advantage that it is self-contained, and is held in place on the paper by weights, and does not mar the paper. The moving part is slightly raised, and does not soil the paper in moving it back and forth. As to the amount of work that can be done with them, a count was recently made of the work done by two men using the Colby protractor (one calling off and one platting) in 25½ hours, and the result shows 216 shots per hour, or 3.6 per minute. About the same speed can be made with the other one mentioned, and this speed considerably exceeded under pressure.

The real test of the accuracy of stadia work is the map itself, and the truth with which it represents the features of the ground covered by the survey. It is said of the engineer officer in charge of the survey of the Great Lakes, that it was his custom to take a chart to the ground which it represented, and picking out three or more well-defined points which were in line, would insist that a line drawn with a straight edge should pass through the same points as represented on the map. This would be a very rigid test, and one that, under ordinary conditions, could not be applied.

The usual way of stating the degree of accuracy attained by stadia work is to give the ratio of the amount of error in closing on a point of known position to the distance as measured through various courses from a preceding known point. This degree of accuracy will vary widely, as will the degree of accuracy obtained in chaining vary widely, when done by different men, for different purposes, and under different conditions. The usual way of stating the error of work where a number of checks have been made is to give the algebraic sum of all closures, and as the signs of errors vary, they tend to balance each other, and the result may show a proportion running into large figures, while individual errors may have been very large. While this method is fair enough if understood, it does not give a true idea of the actual accuracy of the work. The real value is a mean of all closures of short portions, and giving each equal weight.

From Mr. Colby's paper, I find the average closure as obtained over 24 courses between located points to be 1 in 667. The errors in this case are all in the same direction with regard to the direction of the lines, and I understand that since the paper was published, it has been found that the rod interval should have a correction of about 1 in 600 in the opposite direction. If this correction were made to the calculations, the errors would be very small. Mr. Van Ornum gives the average error of the work on the Mexican Boundary survey as 1 in 949. This work covers 182 miles of line, and is the average of 32 checks.

Mr. Jolly, Assistant Engineer, Sewer Department, gives me the following table of 17 checks on work done in St. Louis during 1895. The average is 1 in 2390. On one course the closure is abnormally high, being 1 in about 15,000. Leaving this course out, would give an average of 1 in 1540, which is probably nearer the true degree of accuracy.

TABLE OF ERRORS IN CLOSURE.

Topographical Survey of St. Louis.

By E. J. Jolly, Assistant Engineer Sewer Department, 1895.

No.	From	To	Number of Courses read.	Average Length of Course.	Total Distance.	Error.	Proportional Error.	Remarks.
				M.	M.	M.		
1	△ St. Cyr.	□ 4031	9	265	2383.6	0.90	1 : 2648	These courses are approximately in a straight line, and errors are due to distance almost entirely.
2	□ 4031	△ Robinson.	5	300	1501.5	0.98	1 : 1533	
3	□ 4032	□ 4036	3	298	893.1	0.72	1 : 1240	
4	□ 4036	Base Stone.	4	216	866.2	0.84	1 : 1031	
5	□ 4036	□ 4043	3	187	560.1	0.71	1 : 789	
6	□ 4044	City Limit, 174.	6	249	1493.4	1.73	1 : 863	
7	□ 4038	□ 4033	5	237	1186.5	0.33	1 : 3595	
8	△ Conduit	△ St. Cyr.	14	310	4332.2	0.29	1 : 14938	
9	□ 3962	□ 2795	13	273	3549.6	3.28	1 : 1081	
10	□ 3962	□ 4061	4	121	483.6	0.69	1 : 750	
11	□ 4060	□ 4016	13	341	4437.4	2.45	1 : 1811	
12	□ 4082	□ 4094	4	281	1125.7	0.32	1 : 3474	
13	□ 4075	□ 4084	5	185	925.8	0.96	1 : 960	
14	□ 4086	□ 4085	3	215	646.4	0.69	1 : 941	
15	□ 4085	□ 4095	5	234	1168.3	0.61	1 : 1918	
16	□ 3967	□ 3966	2	228	457.3	0.69	1 : 663	

The average error of work done on the Missouri River during field season of 1895, covering 220 miles of river and about 425 square miles of topography, is 1 in 1004. This average covers all checks, 636 in number, and includes circuits which start and close on stadia stakes, the position of which had been adjusted and may have been somewhat in error. The average closure of 116 lines run between triangulation stations is 1 in 1390. The average length of these lines is 7,109 feet.

The above average is the result of 1,650 miles of main stadia line measured by four observers.

All the above results are obtained by computing the co-ordinates of stakes, and represent errors of field work only. The above results are not given to invite comparison, as they are not comparable. They are results of work done under widely different conditions and for different purposes, and each represents a degree of accuracy which was probably all that was required or desired for the purposes for which the surveys were made. They are not given as examples of what can be or should be done with the stadia, but simply as a few instances of what has been done.

I have endeavored to obtain some records of the accuracy attained by chaining or measuring with steel tape, as is usually done over rough ground and through the brush and weeds ordinarily encountered, but as far as I have been able to find, information on this subject is extremely rare, and such records as I have found are those of work done with more than ordinary care. Mr. Van Ornum, in his paper referred to, gives the average error of five tests with chain as being 1 in 1436, and this after a correction of dropped chain lengths which were detected by the stadia, had been applied. Professor Baker, in *Engineering News*, October 3, 1895, gives a few instances of accuracy in chaining. He gives the average error on public land surveys as 1 in 500; discrepancy between preliminary and location surveys on At, T. & S. F. R. R. as 1 in 2500, also error in chaining by students as from 1 in 400 to 1 in 1000.

My own experience and that of others leads me to believe that with proper appliances an experienced observer with reasonable care can easily obtain an average accuracy of 1 in 1200 to 1500, which, though it may not be as accurate as chaining *can* be done, is, I believe, more accurate than ordinary chaining *is* done, and is certainly well within the limits of errors discoverable on maps of ordinary scales.

As to the degree of accuracy attained in carrying levels by the use of vertical angles, there seems to be very little information printed. My own experience has been that with a level attached to vernier arm of vertical circle, elevations can be carried with an error not exceeding .5 foot for all distances. The long distances showing less error as they tend to balance. A stadia line run last summer about 15 miles in

length and over which levels were run, checking on each stake, showed discrepancies between consecutive stakes of as high as 0.2 foot, the total error for the 15 miles was less than one foot. The average closure of 123 circuits run in 1895 by four observers using instruments without vernier levels, and depending on plate levels alone, was 0.52 foot, the average number of stakes or courses in each circuit is 7.8.

The cost of stadia surveys varies as widely as the number of surveys made. The topographical survey of Baltimore cost for topography alone, excluding triangulation and precise levels, about $1.50 per acre. The scale of the map of this survey is 200 feet per inch, and all buildings, streets, alleys, etc., are located.

The cost of the topography of the survey of St. Louis is given by Mr. Colby as 73 cents per acre; scale of map the same, but few buildings and few street corners were located. A topographical survey of about 3000 acres in the vicinity of Madison, Ill., made in the winter of 1893, by the writer, at a cost of 50 cents per acre, including mapping, scale was 400 feet to the inch, and all buildings, fences, railroads, etc., were located. Several different tracts of land in the vicinity of St. Louis, covering from 100 to 200 acres, have been surveyed at a cost of from 20 to 40 cents per acre. In these cases a scale of 400 feet per inch was used, and contour interval of 2 feet, and only the configuration of the ground determined. A survey of about 9300 acres was made by the writer in Southwest Texas, during the summer of 1894, and the map was made on scale of 400 feet per inch, contours 2 feet apart. Ground was partly covered with brush, and was rolling, though not much broken. Circumstances were favorable for doing rapid work. The cost of completed map was a little less than 7 cents per acre.

Topographical work on the Mississippi River, in 1891, cost about $36.00 per square mile; on the Missouri River, in 1895, a little less than $31.00 per square mile, or from 5 to 5½ cents per acre. This work is platted on scale of about 1000 feet to the inch. Contour intervals are 5 feet, and all buildings, roads, fences, limits of culture, etc., are located. This cost does not include mapping except such field plats as are made in the field, but does include a system of tertiary triangulation on which the topography is based.

Nearly every writer that I have consulted on the subject of stadia surveying, has enlarged more or less on the advantage of the use of the stadia, and the adaptability and flexibility of this method of surveying, so that little remains for me to say in this respect. As the opinion of those who have used it extensively seems to be unanimous as to the desirability of its use, it is a constant wonder to me that it is not in use more extensively. For use in the preliminary work of locating, estimating and reporting on various proposed enterprises of improving land

for subdividing, drainage systems, railroads, canals, irrigation systems, etc., it offers a method of securing desirable and necessary information cheaply, accurately and rapidly, which cannot otherwise be obtained except by laborious and expensive methods.

Specifications for Topographical Theodolites made for Mississippi River Commission, 1895.

A good topographical instrument must possess the following points:

1. It must have an exceptionally good telescope for reading the stadia rod.

2. It must have a good vertical circle, with a delicate level attached to the vernier arm, for carrying levels by means of vertical angles.

3. It must have a good horizontal limb for carrying azimuth.

4. It must be so made as to admit of being easily and firmly clamped to the tripod, so it can be carried with safety.

5. The horizontal and vertical circles should read the same and should be divided into the same number of divisions.

These are the most important features, mentioned in about the order of their value.

1. *General Style of Instrument.*—The general style of instrument as regards plates, compass box, foot-screws, shifting head, etc., should be similar to the Buff & Berger transits of latest patterns.

2. *The Limb.*—This should be 6 inches in diameter and should be divided on solid silver into twenty minute spaces. The divisions should be numbered from 0 to 360 degrees, with figures at each 10 degree division, the numbers increasing to the right, similar to a watch-dial (see Fig. II, Keuffel & Esser Catalogue, page 284). The marks should be deep, full, smooth and distiuct and the whole lacquered so that the limb will not tarnish. The limb should be entirely covered and have suitable glass plates through which to read the verniers, substantially as used in the Buff & Berger instruments. The clamp and slow-motion screw will be of same style as Keuffel & Esser's latest patterns.

3. *The Verniers* should be set at points 45 degrees from the line of sight, to the left of the eye-piece and right of objective, and the vernier nearest the eye-piece should be marked A and the other one B. The verniers should be divided in such a way that 39 parts on the limb will equal 40 parts on the vernier, and read to half minutes. The zero should be at the right-hand end and the five minute divisions should be numbered to the left. Each vernier will be provided with a ground glass reflector suitably placed. The graduation marks should be deep, full, smooth and distinct.

4. *The Compass.*—The compass circle will be divided to one-half degrees and the graduations will be numbered from 0 to 360 degrees and

from right to left, or the opposite direction of that of the limb. The circle should be large enough to permit the use of a $4\frac{1}{4}$ inch needle. This needle, its mountings and bearings, will be of the very best quality. It will be provided with suitable lift for raising needle free from pivot. The compass box will be provided with suitable glass.

5. *The Levels on Vernier Plates.*—The vernier plate will be provided with two levels placed at right angles to each other, one at foot and outside of telescope standards, parallel to telescope and one parallel to axis of telescope under objective end of telescope. These levels will be about 3 inches long and ground to a curve of about $\frac{3}{8}$ of an inch to one minute of arc. The frame will be provided with suitable adjusting screws and be rigid enough to hold the level adjustment with rather severe usage. They must not project beyond the upper plate.

6. *The Telescope.*—The telescope should be provided with exceptionally good lenses so as to secure ample illumination, sharp definition and a flat field. The lenses should be free from chromatic and spherical aberration as far as practicable. The objective should have a clear aperture of $1\frac{5}{16}$ inches and focal length of $12\frac{1}{2}$ inches, and should be carefully centered. The eye-piece will be inverting and be well centered and mounted so as to be easily focused on the cross-hairs and at the same time be held in place so it cannot be lost. The focal length of the eye-piece should be about $\frac{5}{8}$ of an inch, with such magnifying power as will give the best results (see Wurdeman Theod., 154). The Steinhill lense may be used if it materially improves the telescope without a large increase in cost. The telescope will have suitable focusing screw for objective, with proper rack motion which will move with little friction and no slip or lost motion. It will be provided with proper dust-cap and sun-shade. The diaphragm bearing the cross-hairs will be held in place by four adjusting screws, as is usual in the best transits. The cross-wires will consist of very fine smooth spider-web. There will be three horizontal wires and one vertical wire at right angles to the three wires. The intersection of the middle horizontal wire and the vertical wire will be in the center of the diaphragm. The distance between the extreme horizontal wires will be such that they will subtend on a rod a length of one foot at a distance of 120 feet. The intervals between the wires should be equal. All wires will be firmly fastened to the reticule or diaphragm, unless a satisfactory adjustment can be provided for stadia wires. The vertical motion clamp and screw will be in accordance with best practice.

7. *The Vertical Circle.*—The limb should be a full circle, 5 inches in diameter. The circle should be stiffened with a rib. (See Keuffel & Esser Catalogue, 1895, page 276.) The graduations should be on solid silver and be numbered from 0 to 90 degrees in both directions, starting

from two initial points, 180 degrees apart, and in a horizontal plane. (See Fig. 1, Keuffel & Esser Catalogue, page 284.) The circle will be divided into 20 minute spaces. Two double verniers will be provided. They will be so divided that 40 parts on the vernier will equal 39 parts on the limb, and read to half minutes. The verniers should be numbered at 5 minute intervals from zero each way to 20 minutes—the zeros of the two verniers being in a horizontal plane when instrument is leveled up. The horizontal vernier arm should bear a level about 3½ inches long, having a carefully ground glass with a curve of about ⅝ of an inch to a minute of arc. The level will be firmly fastened, and be provided with suitable adjusting screws at each end of the tube.

All graduations will be deep, full, smooth and distinct. Mere scratches on the surface will not answer.

8. *The Wyes*—These will be firm and substantial, and preferably only high enough to permit a telescope movement of, say 30 degrees in a vertical plane above and below the horizontal. A wye 4 inches high and with a 3-inch base would answer.

9. *The Foot-Screws.*—The instrument will be provided with four foot-screws covered with dust-caps at upper extremities.

10. *The Tripod.*—This will be provided with shifting head for quick centering, according to best practice. The legs of tripods will be 5 feet long.

11. *The Packing Box.*—The packing box will be made of white pine, a full inch thick, with dove-tailed corners, and the whole well oiled outside and in. The instrument will be screwed to a suitable base-piece so arranged to hold the instrument firmly in the box without additional packing. The box will be provided with suitable receptacles for plumb, bob, screw-driver and adjusting-pin and reading-glass 1½ inches in diameter, and these articles will be included with instrument. The box should be provided with lock and key.

12. The entire instrument will be made and finished in the best workmanlike manner throughout, and all material must be of the best. Where practicable, drawn or rolled metal will be used instead of castings for such parts as require rigidity.

DISCUSSION.

J. L. Van Ornum.—The writer cannot let the opportunity pass of commending this valuable paper, giving the result of the personal experience of Mr. Maltby with the stadia. It is always necessary to supplement theory with practice, and the experiences of the latter form

the most ready guide. Mr. Maltby has had a wide experience, not only in topographical surveys as such, but also in topography for special purposes (as the planning of irrigation systems), and commends thoroughly the use of the method in such cases. This coincides with the practice of Elwood Mead, State Engineer of Wyoming, who told the writer that he always used the stadia on surveys for irrigation projects. A member of this club has recently made a railroad preliminary survey by the same method. Its application in this field seems very advantageous over the old linear survey, and the result of that survey would make a valuable contribution, if the field location has been made so as to allow direct comparison with the topograpical preliminary.

While the one great argument for the stadia system lies in its ready application because of its making use of the ordinary transit and of very simple methods of observation, reduction and plotting, the fact should be thoroughly appreciated that it is capable of a still greater accuracy when occasion requires it. Those who have used the system can appreciate the advantages in this line, to be derived from the improved instrumental equipment mentioned in the paper. The dividing of the rod in true unit divisions (as also mentioned) comes into especial usefulness where greater accuracy is desired, as it enables the observer to read the vertical angle to the exact point of the rod at which he reads the distance, and so exclude the error caused by changing the pointing before reading this angle; it gives true distances on side-sights as well as station distances by causing the derived interval factor to change with the interval. Besides these advantages, those who have used the system count many others.

The writer cannot see the advantage of making the stadia-rod symmetrical about the center. He has used both rods so designed and rods having figures to aid in their reading, and it is his experience that the latter is more readily read. Of course, the use of figures necessitates the holding of one certain end always at the ground, but he has found a mistake in this regard extremely rare among rodmen. To reassure those who fear its occurrence, would it not be feasible to have the nail in the shoe of the rod (so generally used to aid in holding it vertical) placed only in one end, that end being the bottom?

The advocacy of sketching seems just. When it is considered that the purpose of topography is to show the surface as correctly as practicable from a framework consisting of comparatively few located points, showing the natural curves and gradual changes of slope instead of angles and abrupt changes that the rigid method requires, it seems desirable to indicate just the existing condition of these curves and slopes. Perhaps it could be described, but not as well, and the advantage of relieving the attention of the engineer of all the multiplicity of details

is applicable here. Instead of having to bear in mind a dozen details to be described in one way or another, he has only to make his sketch as he overlooks the ground, and these details appear without especial thought. Nor is it the writer's experience that it takes longer when one is accustomed to sketching than does a description, and it is more complete. Descriptions are not to be disregarded, for often they are most useful. But, as a rule, sketches seem desirable and at times a necessity. The method of sketching on detached sheets, as described in the evening's paper, seems excellent at times; still, circumstances will often put the sketches in the field-book, thus reducing the field-equipment and keeping all the field-notes together.

Methods and practice vary under differing conditions and with different men. In some cases it is desirable that it should be so; in others they may be improved upon. Thoughtful comparison and discussion will point out that which is best, and leave behind what is not well proved.

B. H. Colby.—Mr. Maltby's paper seems to me deserving of much commendation. It is practical from beginning to end. The reader is never in doubt as to the author's opinions or as to the reason for the faith that is in him. I am glad to see another experienced man come along and join the ranks of those who believe that it takes first-class instruments to secure first-class work.

Give a surgeon a sickle to amputate a leg, and he would not be more handicapped than a surveyor who is equipped with the ordinary transit and leveling-rod in making a topographical survey. To do good and rapid topographic work a stadia *board* at least 5 inches wide must be used. As well try to cut down trees with a jack-knife as to use a leveling-rod in taking topography.

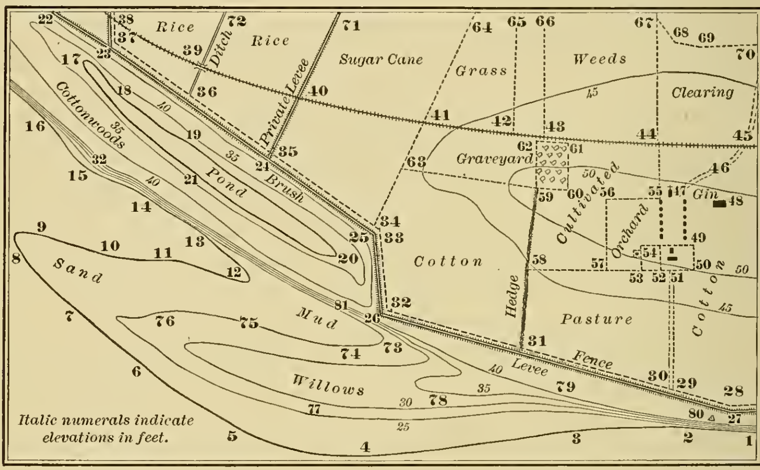
The points noted about the theodolite to be used are excellent. One of the most essential, to my mind, is that the vernier arm of the vertical circle shall have a level, attached to it and movable with it. With this attachment levels can be carried by vertical angles and stadia distances, when read both ways, almost as accurately as with the ordinary Y-level, and accurately enough for the requirements of any topographic survey. I think Mr. Maltby will agree with me in saying that levels can be run in this way within one-tenth of a foot to a mile. When the needle is used and distances and vertical angles are not read both ways, levels cannot, in my judgment, be depended upon unless the telescope is reversed at each station and the vertical angle read in both positions. The magnetic needle has had its day and should no longer be placed upon an instrument used for surveying.

I think the present high degree of accuracy obtained with the

stadia, could be materially increased by increasing the size of the telescope used.

I believe the time will surely come when the stadia will be used upon all railroad surveys, including preliminary lines and final estimates.

The good results obtained by Mr. Maltby are to be expected whenever and wherever such methods are followed. I am heartily in accord with nearly all the opinions and views expressed by Mr. Maltby, but I cannot agree with him in one detail, that of making field sketches. I would let his remarks in this regard pass unanswered were it not for the fact that he quoted me as having once written that: "Sketching is entirely useless, and worse than useless, as it is a waste of time." What I said in the paper quoted from is: "It seems to me that the time



an experienced topographic engineer spends in sketching is almost wasted."

Now I believe that whenever a surveyor is able to call off his "shots" to the recorder, using a nomenclature sufficiently comprehensive to enable any one at any subsequent period to accurately plat and properly connect with each other all the points and lines of the survey, that surveyor should not make a sketch. If he is not master of such a system of taking notes he is obliged to sketch to make his notes intelligible. I do not believe that there is any piece of ground, improved or unimproved, that cannot be surveyed, by stadia, and all of its features accurately located and mapped without making a sketch. The whole secret of topographic surveying without sketching is in taking the notes. A comprehensive system of nomenclature must be used. Such a system is very easy to acquire.

After making these statements it seems but fair to give some examples of the chief features of such a system, one that has been tested and found satisfactory in practical use. For a practical illustration I have taken a part of a topographic map* published by the Mississippi River Commission and furnished me by courtesy of Mr. J. A. Ockerson. On this map I have printed numbers from one to eighty-five inclusive, indicating different points located by stadia. These points would be designated in the "Object Column" of the note-book as follows:

1. Shore river, foot of bank.
2. Shore river, E. end sand bar.
3 to 10. Shore river, sand bar.
11. Shore river, sand and mud bar.
12. Shore river, mud bar.
13. Shore river, W. end mud bar.
14. Shore river, E. end sand.
15. Shore river, sand.
16. Shore river, W. end sand.
17 to 21. Water surface pond, cottonwood and brush.
22. Top levee, trees both levee sides.
23. Junction levee and levee No. 2, S.E cor. trees.
24. Junction and private levee, trees and brush S. of levee.
25. Levee bends, trees S.
26. Levee bends, trees end.
27. Levee bends, grass S., cotton N.
28. Fence bends, cotton.
29. Fence E. S. road, cotton.
30. Fence W.S. road pasture.
31. Fence S. end hedge, pasture N. & E., cotton N. & W.
32. Fence bends, cotton.
33. Fence bends, W. edge cotton, E. edge sugar cane.
35. Fence & private levee, sugar N.E., rice N.W.
36. Fence and ditch 4′ wide, rice.
37. Fence bends, rice N.E.
38. Fence & R.R., rice E.
39. Center R.R. bridge (60′ x 30′) and ditch 4′ wide.
40. Center R.R. and private levee, rice W., sugar E.
41. Center R.R., sugar W., grass E.
42. Center R.R., grass W. & S., deadening E. & N.
43. Center R.R., deadening W., weeds E.
44. Center R.R. weeds N.W., clearing N.E., cotton S.E., cult. S.W.

* The fine detail of the original is omitted in the reproduction, as being non-essential to the discussion.—*Secretary, Ass'n of Eng. Socs.*

45. Center R.R. and road No. 2, clearing N., cotton S.
46. Center road No. 2, in cotton.
47. Center road No. 2, S. end, cotton.
48. N.E. cor. gin. (30′ x 75′).
49. N.E cor. d.y. fence, cotton.
50. S.E. cor. d.y. fence, cotton.
51. D.y. fence E. side road, cotton.
52. D.y. fence & cross fence, pasture.
53. S.W. cor. d.y. fence, pasture S., orchard N.W.
54. N.W. cor. d.y. fence, orchard W. of N.
55. N.E. cor. orchard, cotton N.E., cotton N.W.
56. N.W. cor. orchard, cultivated N. & W.
57. S.W. cor. orchard, cult. N.W., pasture S.
58. Hedge, cult. N.E., pasture S.E., cotton W.
59. S.W. cor. cemetery fence, N. end hedge, cult. S.E., cotton S.W., grass N.W.
60. S.E. cor. cemetery fence, cult.
61. N.E. cor. cemetery fence, cult.
62. N.W. cor. cemetery fence, grass.
63. Grass N.E., cotton S.E., sugar W.
64. Edge trees, grass S.E., sugar S.W.
65. Edge trees, deadening E., grass S.W.
66. Deadening W. & N., weeds S.E.
67. End fence, weeds W., cane and trees E., clearing S.E.

68–70. Fence bends, cane and trees N., clearing S.

71. Private levee bends, trees N.E., sugar E., rice W.
72. Ditch (4′) rice.
73. Mud and sand bar.

74–75. Mud and sand bar, edge willows.

76. Sand bar, end willows.
77. Sand bar, edge willows.
78. Sand bar, end willows.
79. Sand bar and grass.
80. Top bank, grass.
81. Top bank sand and mud bar, edge cottonwood trees.

By use of such a system as this the proper connections can be made between all the points of the survey with perfect accuracy. Abbreviations can be used to a much larger extent than I have indicated.

J. A. OCKERSON.—Mr. Maltby has made a valuable contribution to the literature relating to the use of the stadia from a practical standpoint, in the paper presented to this Club. The theorist and the novice have given too much advice as to the use of the stadia. Most of them discover new devices for rods or new eccentricities of refraction, and between them lies largely the responsibility for the limited use of this most valuable method of making all classes of surveys.

Those of us who are familiar with the stadia and its great utility cannot be frightened by the hobgoblins conjured up by visions of "differential refraction." But the beginner will certainly hesitate to adopt an instrument which he is told gives one result in the morning and a different one at noon; one result on a cloudy day, another when the sun shines. This is true only in a limited sense. Before it means anything we must know what the limit of error allowable is. The chain as ordinarily used, gives the distance between two points with a certain degree of precision; with a steel tape a still better determination can be made; the same distance measured with a primary base apparatus will show that both chain and tape are in error, owing to variation in temperature, inclination, etc. This, however, is far from demonstrating that the chain and tape are not useful and valuable to the engineer, although it has been shown that the absolute length cannot be determined by their use. The same is true of the stadia. Because it has been demonstrated that measurements are probably not made with an accuracy of one in ten thousand is no argument against its use where an error of one in one thousand is inappreciable and not important. The stadia has been systematically slandered by holding up to view the fact that errors of small magnitude are continually occurring in measurements made by this method, at the same time obscuring the fact that such errors are wholly inappreciable in perhaps ninety per cent. of the work the surveyor has to do.

Those of us who have had the stadia in constant use for a quarter of a century, under a great variety of conditions, from the Great Lakes to the Gulf of Mexico, know, beyond the shadow of a doubt, that it is the only rational instrument to use in topographical work; that the degree of accuracy is well within the limits of errors in platting and change of scale in the paper itself; that attempts to go into refinements beyond these limits are more than useless, because the manipulation of rods and instruments in efforts to secure this imaginary increase in accuracy. tends to strip the stadia of its greatest charm and utility and add to the difficulties and expense of using it.

As an economical and expeditious method of locating points and obtaining their elevations for any purpose, whether it be for a topo-

graphical map, location of a railway or an irrigating canal, it stands without a rival.

The assertions here made as to the accuracy of stadia work cannot be successfully controverted, as they are amply verified by thousands of comparisons which may be found in our note-books covering a long period of time and a great extent of work.

The impossibility of defining a recognizable condition of atmosphere which would give absolutely uniform results in measurements; the infinite variety in atmospheric conditions and the brief existence of any one condition; the absolute necessity of being able to do work under any and all conditions; the small gain in accuracy derived from careful attention to the conditions of the air; all of these simply emphasize the facts that the best use is made of the stadia when the preliminary value of the wire interval is determined at any time when the conditions are not abnormal; that the work should continue daily without regard to whether the atmosphere is boiling or not; and that the final value for the wire interval shall be derived from comparisons of the stadia lengths between fixed points, as measured under all of these constantly varying conditions, with the true lengths as determined by triangulation or other rigid methods.

In the great majority of cases, the preliminary value of the wire interval will answer all requirements.

The cost of topographical work will depend largely on the amount of detail required and the scale on which it is to be mapped.

In the survey of the Mississippi River all features, natural and artificial, that will show on a scale of 1:10,000 are located instrumentally and sufficient elevations are determined to develop contours of elevation 5 feet apart. The shore line of the river, islands, sand-bars, tertiary triangulation, etc., add considerably to the work and should be taken into account when comparing with cost of other work per square mile. About 1,000 square miles of topography lying along both banks of the river from Alton northward and including several cities, like Hannibal, Quincy, Keokuk, Burlington, etc., cost an average of $39.46 per square mile, or about six cents per acre. The number of points located per square mile averaged 371.

Platting the notes and drawing the maps complete on a scale of 1 : 10,000, cost about five dollars per square mile.

Printing an edition of 1,000 copies on a scale of 1 : 20,000 sheets 22 inches by 36 inches inside of border, cost about five dollars per square mile.

Mr. Maltby is entitled to the thanks of the profession for his advocacy of the utility of the stadia for surveys in general, and his experience should endow his views with great weight.

For fear this discussion may be considered as endorsing or advocating loose and careless work, it may be said that the writer is a firm advocate of careful and accurate work, but considers it quite as grave an error to carry the refinements of a work, at the expense of time and money, far beyond all reasonable requirements, as it is to fall a trifle short in such requirements. The work should be just as accurate as the uses to which it is to be put require; no more, no less. The conscientious engineer will strive to come as near this golden mean as practicable. His departure from it will stand as a measure of his skill, judgment and comprehension of the requirements.

Journal of the Association of Engineering Societies, Vol. XVII, No. 4, October, 1896.

Yours truly
J. F. Holloway

Honorary Member and Ex-President of the Civil Engineers' Club of Cleveland.

JOURNAL OF THE ASSOCIATION OF ENGINEERING SOCIETIES, 1896.

ERRATA IN OCTOBER NUMBER.

Page 119, Fig. 3, title: For "Vicensa" read "Vicenza."

Page 132, 9th line: For "Michael Angelo" read "Antonio da Ponte."

Page 138, Fig. 20, title: For "Bridge over the Nydeck" read "Nydeck Bridge over the Aar."

Page 139, last line: For "Maritime Alps" read "Pyrenees."

Page 140, 5th line: For "Morlaux" read "Morlaix."

Page 141, Fig. 23, title: For "Morlaux" read "Morlaix."

Editors reprinting articles from this journal are requested to credit both the JOURNAL and the Society before which such articles were read.

ASSOCIATION
OF
ENGINEERING SOCIETIES.

Organized 1881.

VOL. XVII. OCTOBER, 1896. No. 4.

THE HISTORICAL DEVELOPMENT OF STONE BRIDGES.

BY PROF. GEORGE F. SWAIN, PRESIDENT OF THE BOSTON SOCIETY OF CIVIL ENGINEERS.

[Lecture delivered before the Society, June 17, 1896.*]

THE first stone bridges were simply blocks of stone laid horizontally over an opening. Such bridges are still used for spanning narrow openings, and in some countries for crossing streams of not inconsiderable width. In buildings, this is still the most common mode of spanning an opening. The next step was to corbel out, letting one stone project beyond the one beneath it, and in this manner spanning larger openings than would be possible with one stone alone. The next step was to cut off the lower projecting corners of these corbelled stones, making a structure in appearance like an arch—a so-called false arch—though in principle it is not in any sense a true arch. These false arches were used by the Egyptians, the Assyrians, the Greeks, and other older nations. Fig. 1 shows a Greek structure of this kind.

The principle of the arch appears to have been known to the Assyrians and to almost all of the older nations whose structures have become known to us. All of the Assyrian arches thus far discovered are of brick, the bricks being made of a wedge shape, or thicker at the outside than at the inside. Most of the arches were semicircular, and the maximum span found is 15 feet. The only pointed arch discovered in the Assyrian ruins is built of brick of the ordinary shape, the joints being thicker on the outside; while instead of the keystone, ordinary bricks were laid longitudinally between the two sides. The arch was used by

*Manuscript received September 29, 1896.—*Secretary, Ass'n of Eng. Socs.*

the Assyrians for doors, gates, drains, aqueducts, and chambers or galleries.

Still earlier than the Assyrian arches, which belong to about the ninth century B.C., were the brick Babylonian arches, as early as 1300 B.C., while in Egypt arches were known as far back as the fifteenth century B.C. The early Egyptians, however, though undoubtedly acquainted with the arch principle, avoided arches, and seem not to have fully grasped the idea of using a large number of thin stones laid with the long sides touching. The Hindoos to this day refuse to use arches. They have a saying that "the arch never sleeps," meaning by this that it continually exerts a horizontal thrust upon its supports, tending to disintegrate and destroy them. This quaint saying is true and suggestive. The arch will not withstand the ravages of time as well as the structure which causes only vertical pressures upon its supports. Any yielding, decay or defect may result in the collapse of the entire structure. The Egyptians, then, who

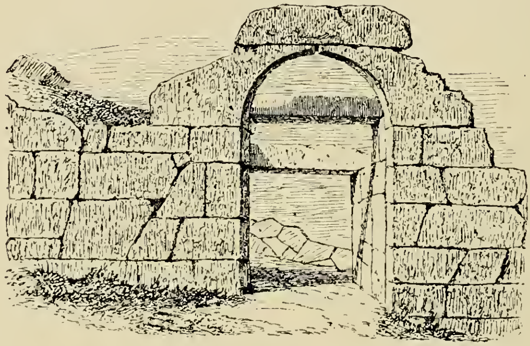


Fig. 1.—Greek False Arch.

also attached weight to this principle, never applied the arch on a large scale, or to large edifices.

To the Romans is due the special development of the arch, and its application on a large scale in the construction of viaducts, aqueducts, sewers, buildings and bridges. One of the earliest of their arches was the Cloāca Maxima, a sewer which is still in use and in good condition. The arch consists of three rings of stones, and its construction shows a perfect appreciation of the principles involved. The Romans built many arch bridges over streams, one of the bridges over the Tiber having a span of 84 feet; while the Emperor Augustus constructed a bridge of five spans over the river Marachia at Rimini. Between Rome and Gābii there was a viaduct with nine arches. Fig. 2 shows one of the early Roman bridges across the Tiber, while Fig. 3 shows another typical Roman bridge of three

spans at Vicenza, and Fig. 4 still another early Roman bridge, with triumphal arches over the roadway at each end.

The ancient Romans, while they built a large number of bridges,

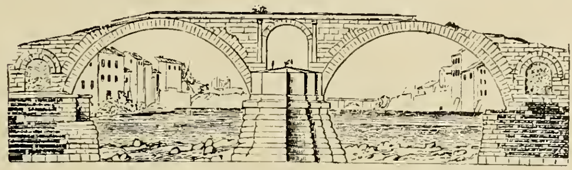


FIG. 2.—FABRICIUS BRIDGE AT ROME.

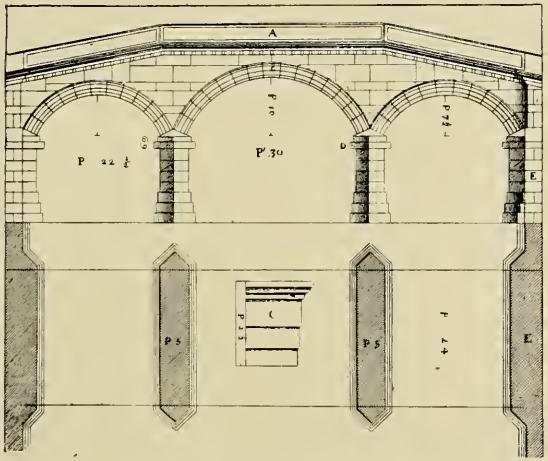


FIG. 3.—OLD ROMAN BRIDGE AT VICENSA.

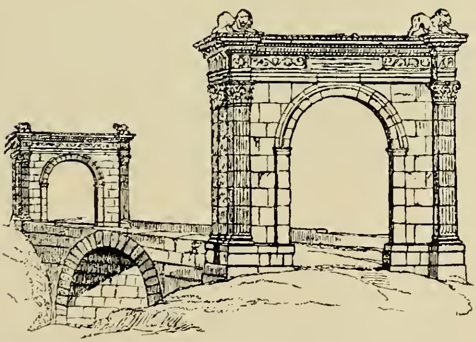


FIG. 4.—BRIDGE OF ST. CHAMAS.

aqueducts, and other works of this nature, met with difficulties in the foundations of their bridges over streams. Where they were able to build upon a rock foundation they constructed works which have en-

dured, and by giving considerable width to the piers of their bridges across streams or on soft ground, they succeeded in meeting temporarily the difficulties attendant upon a foundation on compressible material; but they were not able to protect themselves sufficiently against the effect of floods, and the excessive width of their piers, by diminishing the waterway, greatly increased the danger of undermining, which has been the cause of the destruction of most of their bridges. The piers, whose thickness was, on an average, one third of the span, were not carried sufficiently deep; the riprap or stone filling by which they were protected was not effective; in a word, their foundations were imperfect.

The arches of the Roman bridges were almost always semicircular; in exceptional cases they adopted flattened curves, whose rise was never made less than one third of the span. In general the extrados was parallel to the intrados, and the thickness of the arch ring at the crown was on an average about one twelfth of the span.

The number of arches in the Roman bridges was generally uneven, and the central arch had a larger span than the others. Often the spans decreased progressively from the center of the bridge to the ends. Under these conditions the roadway above could only be made horizontal, with the same depth of filling above each arch, when the springings of the arches were placed at different levels, and higher towards the ends. In general, however, they preferred to place the springing lines of all the arches at the same level, making the roadway inclined from each end towards the center, sometimes with an angle at the center, and sometimes with a horizontal portion above the center span. The contraction of the water-way due to the thickness of the piers was in part made up for, in some cases, by openings through the masonry above the piers or in the haunches. (Fig. 2.) But while the excessive thickness of the piers of the Roman bridges had the effect of increasing the danger of undermining, it resulted in the advantage that each pier was able to resist the thrust of the arch on either side of it, even though the arch on the other side should be destroyed. The fall of one arch, therefore, did not result in the fall of the adjacent arches, as would be the case in many modern bridges. The piers were made triangular at each end, or sometimes semicircular. The bridges carrying roads were provided with continuous solid parapets.

In the arches, as well as in the other masonry structures of the Romans, the stones were laid dry, that is, without beds of mortar. Moreover, each arch was frequently made up of several rings of arch stones not bonded, or connected together in any way. Some of these bridges are still in existence and differ comparatively little from modern structures. Many of these Roman bridges were adorned with statues on the piers and on the approaches.

The most important of the Roman bridges, however, were in connection with the aqueducts which supplied the cities with water. There were nine of these aqueducts supplying ancient Rome in the time of Frontinus, "Curator Aquarum" from 97 to 106 A.D. The first was underground and was built by Appius Claudius 312 B.C. The second was also underground and was built forty years later. The third, or Martian aqueduct, was built 144 B.C., by Quintus Martius, and was partly

FIG. 5.—CLAUDIAN AQUEDUCT CROSSING THE CAMPAGNA, AT ROME.

above ground, and its remains may still be seen. It had nearly 7,000 arches in a course of 39 miles, with spans of 16 feet. It was constructed of different kinds of stone, red, brown and yellow. The arches in many places were more than 70 feet in height. This aqueduct was built so strong that the two succeeding ones were built on top of it, thus giving three tiers of arches, one above the other. The sixth aqueduct was constructed by Agrippa, in 33 B.C. The seventh was built by Augustus. The eighth, or Claudian aqueduct, 45 miles long, and the ninth, 62 miles long,

were both begun by Caligula, A.D. 38, and completed by Claudius, A.D. 52. Along the greater part of their course they are carried on the same line of lofty arches with spans of about 20 feet, the highest of all the aqueducts supplying Rome. Magnificent remains of the Claudian aqueduct, built of massive blocks of tufa, still exist for many miles across the Campagna. (Fig. 5.) "A great number of these arches are still in good preservation, with, in places, later arches built under them by Severus in 201, probably to support them after injury by an earthquake." (Middleton.) Two other aqueducts were built later, the Aqua Trajana, built by Trajan, A.D. 109; and the Aqua Alexandrina, built A.D. 226, by Severus Alexander.

In addition to the aqueducts which supplied Rome, the Romans built many other aqueducts, among which may be specially mentioned that at Nismes, in Southern France, and those at Segovia and Tarragōna, in Spain. The noted Pont du Gard (Fig. 6) was built in the aqueduct supplying Nismes, and is one of the earliest aqueducts constructed by the Romans outside of Italy. It is supposed to have been built in the time of Augustus. This aqueduct had three tiers of arches, but only one channel at the top. The length of this bridge at the top of the second tier is 885 feet, and its maximum height over the river Gardon is about 160 feet. The arches of the two lower tiers are semicircular. The large arch, through which the river passes, is 80 feet 5 inches in span; the three on the right side of this are 63 feet, and the smaller ones 51 feet; the arches of the upper tier are all equal in span, 15 feet 9 inches. The thickness from face to face is at the first story 20 feet 9 inches, at the second story 15 feet, at the third 11 feet 9 inches. The depth of the keystone of the large arch is 5 feet 3 inches; that of the others 5 feet, while those of the upper story are 2 feet 7 inches. The lower arches are formed of four separate rings, the next above of three, and the upper of one. The arches thus consisted of separate narrow arches side by side, not bonded or connected together. This structure is constructed of freestone with rubble filling in the piers and spandrels. The stones were laid without cement, and projecting stones were left to support the centers. The dimensions of the channel are 4 feet wide and 4 feet 9 inches high. Above the small arches of the upper tier cement was used in the rubble masonry about the channel. This cement has become as hard as the stone itself, forming one impermeable mass, and preventing any filtration. This beautiful structure was partly destroyed at the ends, at the beginning of the fifth century, by the barbarians who besieged Nismes. In 1743 it was repaired and the piers prolonged to carry a new bridge. The entire length of the aqueduct of which this bridge forms a part is over 25½ miles. The fall given to the water along the entire length is 0.04 feet per 100 feet, and is uniform throughout. This

great work of engineering will compare favorably with any of modern times. It shows that the Romans understood thoroughly the art of leveling, and much more of the science of hydraulics than we generally credit them with. Considering the state of physical science at that time, the skill and care displayed in this and other similar works is little short of marvelous.

The aqueduct of Segovia, Spain (Fig. 7), was built by the Emperor Trajan, and is of squared stone laid without mortar, crossing a valley

FIG. 6.—PONT DU GARD, AT NISMES.

with a length of more than 2,500 feet. It is in many places nearly 100 feet high. This aqueduct has 109 arches, of which 30 are modern, but like the old ones. It has carried water up to a very recent date, and possibly may still be in use. The aqueduct at Tarragōna is similar, and of about the same height, with 25 lower arches and 11 upper arches.

One of the most famous Roman bridges is the Bridge of Alcantara, across the Tagus near Alcantara, close to the frontier between Spain and Portugal. It is said to have been built by Trajan about 98 A.D., and

had six semicircular arches, with one center span of 115 feet, with a height of 203 feet above the river. The masonry was dry.

Another work which is sometimes described is the so-called mole of Caligula, the remains of which may still be seen in the bay of Pozzuoli, near Naples, and which is thought by some to have extended entirely across the bay to Baie. Thus Palladio, writing in 1570, said: "But among all of the celebrated bri lges, that is recorded as a marvelous thing which Caligula made from Pozzuolo to Baie in the middle of the sea, in length

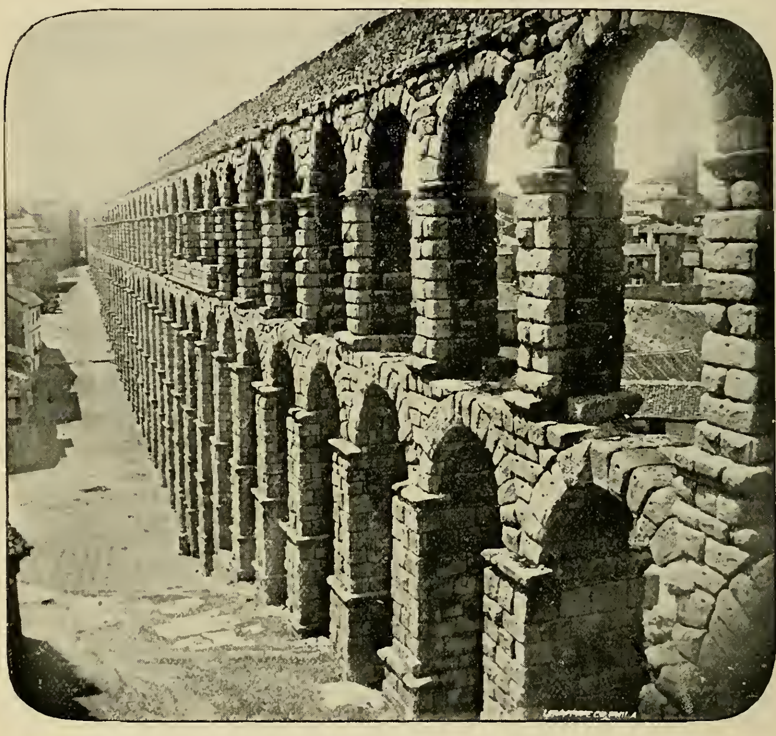


Fig. 7.—Old Roman Aqueduct at Segovia, Spain.

somewhat less than three miles; in which they say he spent all the money of the Empire." The history of this work, however, is imperfect and confused; no one knows who built it or conceived it, and it was already in a considerable state of dilapidation at the time of Hadrian, so that it appears certain that Caligula did not build it or even repair it, but at one time had a bridge of boats constructed across the bay from its seaward extremity, upon which he built a paved roadway similar to the Appian way.

It is evident from these examples that the Romans were masters of the science of engineering so far as concerns the construction of stone bridges and of aqueducts, and history shows that their skill extended to other branches as well.

After the fall of the Western Empire there was little bridge building in Europe until the twelfth century, when the increase of travel, together with the rapid development of cities and of trade, rendered imperative better facilities for crossing streams.

Fig. 8.—Bridge of St. Benezet, across the Rhone at Avignon.

In France a religious association known as "Brothers of the Bridge," was founded by Benedictine monks, and flourished especially during the twelfth and thirteenth centuries. By this order the building of bridges was assumed as an act of piety. They established houses for the accommodation of travelers at the stream crossings, acquired means for constructing bridges, and in some cases superintended their erection. One of the earliest bridges built by this organization was at Durrance, but due consideration not having been given to the waterway, it

was soon demolished by floods. Another, known as the Bridge of St. Benezet (Fig. 8), and the funds for which were obtained by a pretended miracle, was built at Avignon, over the Rhone, having been begun in 1177 and completed in 1187. This bridge is said to have had twenty-two arches and was 2,000 feet long, and only 13 feet wide between parapets. The largest span was about 110 feet, and the arches were segmental. In 1385 Pope Boniface IX had some arches of the bridge destroyed for his own safety, and various mishaps resulted in the destruction of others. In 1410 the inhabitants blew up the tower, which carried down three other spans, and in 1670 the river carried away several more. There are now only the remains of a few arches.

Many other bridges were built by the "Brothers," among which may be named those of St. Esprit, Ceret, Nions, Castellane, Villeneuve d'Agen, and a remarkable bridge at Vielle Brioude, over the Allier, built 1454, with one arch having a span of 183 feet, and a rise of 70 feet. Only the arch ring was of cut stone, and it was quite thin, the rest of the masonry being of rubble. The piers above high-water were faced with stone on the outside, the inside being filled with sand and gravel. This bridge was reconstructed about the middle of the present century.

Of all these bridges built between the twelfth and sixteenth centuries, it may be said that it is remarkable that they stood as well as they did. They were cheaply constructed, and very narrow, seldom 20 feet in width, generally not over 13 to 16 feet, and sometimes but 6 or 7; the piers were very thick, and the spandrels either perforated, or filled with earth.

In several bridges of this period Gothic or pointed arches were used. Thus the bridge over the Ticino, at Pavia, built in the fourteenth century, under Galeas Visconti, Duke of Milan, had seven equal spans of 70 feet with a rise of 64 feet. The piers were about 16 feet thick. This bridge was built of brick, and was covered, the roof being supported by marble columns. It is, however, not now in existence. The present structure is itself several centuries old, and has six arches, the maximum span being 100 feet.

Other notable bridges of the Middle Ages were the bridge over the Danube at Ratisbon, built in 1133, with fifteen semicircular arches varying in span from 33 feet to 53 feet; the bridge over the Elbe at Dresden, built in the twelfth century and restored about 1730, in which the thickness of the piers was almost as great as the spans of the arches; the old bridge over the Moldau at Prag; and the old bridge over the Main at Würzburg. Fig. 9 is a view of the Karlsbrücke at Prag, showing principally the tower at the end. This bridge has sixteen spans, and was built between 1357 and 1507. It was partially destroyed by a flood in 1890. As shown in the picture, it is ornamented by thirty statues and

groups of saints, including a bronze statue, in the center, of St. John Nepomuc, the patron saint of Bohemia, in whose memory the bridge is visited yearly by thousands of pilgrims. The saint is said to have been flung from the bridge in 1383, by order of the Emperor, for refusing to betray what the Empress had confided to him in confessional. The body is said to have floated for some time in the river, with five brilliant stars hovering over the head.

FIG. 9.—KARLSBRÜCKE AT PRAG.

Fig. 10 shows an old bridge at Kreuznach, with buildings over the piers.

Perhaps to us the most interesting bridge built within this period is the old London bridge over the Thames, built by Peter of Colechurch. This bridge was begun in 1176 and finished in 1209, and was 926 feet long and 40 feet wide. It contained a drawbridge and nineteen pointed arches, varying in span from about 9 feet to 20 feet, with massive piers, varying from 25 feet to 34 feet wide. The obstruction caused by these

huge barriers, and the large number of piers, reduced the entire channel of the river from its normal breadth of 900 feet to a total waterway of 194 feet, or less than one-quarter. It is said that this obstruction caused a fall of water at the bridge of about 5 feet. Only eighty years after its completion this bridge was in such bad condition that men were afraid to pass over it, and the houses on top had arches built between them to hold them together. In 1758 the houses were removed, and a large arch constructed in place of two smaller ones. In 1738 the West-

FIG. 10.—ANCIENT BRIDGE AT KREUZNACH, GERMANY.

minster bridge was begun, and completed in 1749. This was the second bridge over the river; but it did not relieve the traffic sufficiently, and repairs were made on the London bridge, at a cost of £100,000. In 1824 to 1831, the new London bridge was built, consisting of five semi-elliptical arches, with two spans of 130 feet, two of 140 feet, and the central one of 152 feet 6 inches, and a rise of 37 feet 6 inches. This was the largest elliptical arch built up to that time.*

* The roadway of this bridge was 52 feet wide. It was built just above the old bridge, and was designed by John Rennie, and built by his sons, John Rennie and Sir George Rennie, at a cost of £426,000.

To the mediæval period belongs also the Ponte Vecchio, over the Arno, in Florence, built originally in 1177, but reconstructed in 1345. It has three segmental arches, with spans of from 85 feet to 94 feet 6 inches, piers 20 feet 4 inches thick, and a depth of keystone of 3 feet 3 inches. Its breadth is 105 feet, and it carries a covered gallery, constructed by the Medici, forming the continuation of a passage from the Pitti palace to the old Ducal palace, with stores on the sides, originally

FIG. 11.—BRIDGE OF ALCANTARA, TOLEDO, SPAIN.

intended for goldsmiths' shops. This is one of the first mediæval bridges which is segmental.

The largest stone arch span constructed up to the present day was found in a bridge built in 1377 by Barnabo Visconti, at Trezzo, over the Adda, which was destroyed in a local war in 1416. It was fortified, and defended the approaches of the castle of Trezzo. It was a segmental arch, with a span of 237 feet and a rise of 68 feet.

In Spain, the Middle Ages saw the construction of several remarkable works. The bridge of Alcantara, over the Tagus at Toledo (Fig. 11).

was built in 997, and has one large span of 93 feet, adjoining which is a small span of 52.5 feet, both semicircular. The tower has a Moorish aspect, but the general character of this structure is Roman.

The St. Martin bridge, over the same river and in the same city (Fig. 12), built in 1203, is still more picturesque. The central arch is pointed, with a span of 132 feet, though the angle at the top is scarcely perceptible. Of the adjacent arches, some are pointed and some semicircular.

FIG. 12.—ST. MARTIN BRIDGE, TOLEDO, SPAIN.

In comparing the bridges of the Middle Ages with those of the Romans, we see that those of the Middle Ages had often much steeper approaches, narrower roadways, and that the spans of the arches of the same bridge were very unequal. The shape of the arches remained, for the most part, as with the Romans, semicircular or but slightly depressed, though sometimes pointed and sometimes segmental, and the piers were very thick and pointed. At the ends were frequently towers

or chapels (Fig. 13), either for purposes of defense or to commemorate the religious origin of the structure. While the workmanship was sometimes good, it was generally coarse and defective to such an extent that it is difficult to understand why some of them have stood until the present day. Some of them were remarkable for length of span, and for small thickness of the arch ring, as well as for small width.

In the sixteenth and seventeenth centuries many bridges were

FIG. 13.—BRIDGE AT TOURNAI, BELGIUM.

built, particularly in France and Italy. After the collapse of the old Pont Notre Dame in Paris, in 1498, the new structure, which still exists, was begun, and completed in 1507, and this was followed by the construction of several other stone bridges in Paris. The Pont Neuf was begun in 1578 and completed in 1604; the Pont St. Michel in 1617, the Pont Marie in 1635, the Pont au Change in 1639, and the Tournelle in 1656. These bridges still exist, although, to provide for the increase in traffic, the Pont St. Michel was rebuilt in 1859 and the

Pont au Change in 1858, in each case the number of spans being reduced and the width increased. The Pont de la Tournelle was widened in 1845 by the addition of cast iron arches.

In Italy, the Trinity bridge at Florence (Fig. 14), built in 1570 by Ammanati, and the Rialto bridge over the Grand Canal at Venice, completed in 1590, belong to this period. The Trinity bridge has three nearly elliptical arches, from 87 feet 7 inches to 95 feet 10 inches in span, and piers 26 feet 3 inches thick. The Rialto bridge, built 1578 by Michael Angelo, has one segmental arch with a span of 96 feet 10 inches, and a rise of 20 feet 7 inches. The approaches are steep, with

Fig. 14.—Trinity Bridge at Florence.

marble steps, and there are stores on each side of a central passageway.

At this time, it will be remembered, the use of very flat arches was considered a bold nndertaking, and the Fleischbrücke (or Pont des Boucheries), in Nüremberg, built in 1599, by Peter Carl, consisting of a single arch with a span of 97 fect, and a rise of only 13 feet, or less than one-seventh of the span, was considered a very bold structure. The thickness of the keystone of this bridge was 4 feet; the abutments were built with joints in continuation of the arch, and were founded upon piles driven obliquely.

In the sixteenth and seventeenth centuries, then, the principal development appears to have been in the increasing use of elliptical, seg-

mental, or other flattened curves instead of semi-circular arches, and in the increased care devoted to construction and to ornament. Worthy of note, too, is the improvement gradually being made in the foundations of bridges. The structures of earlier date had generally been founded on stone filling, and the piers had been necessarily thick, in order to distribute the load. The later bridges were founded upon pile platforms, or in caissons; while in the bridge over the Maas at Masstrecht, begun

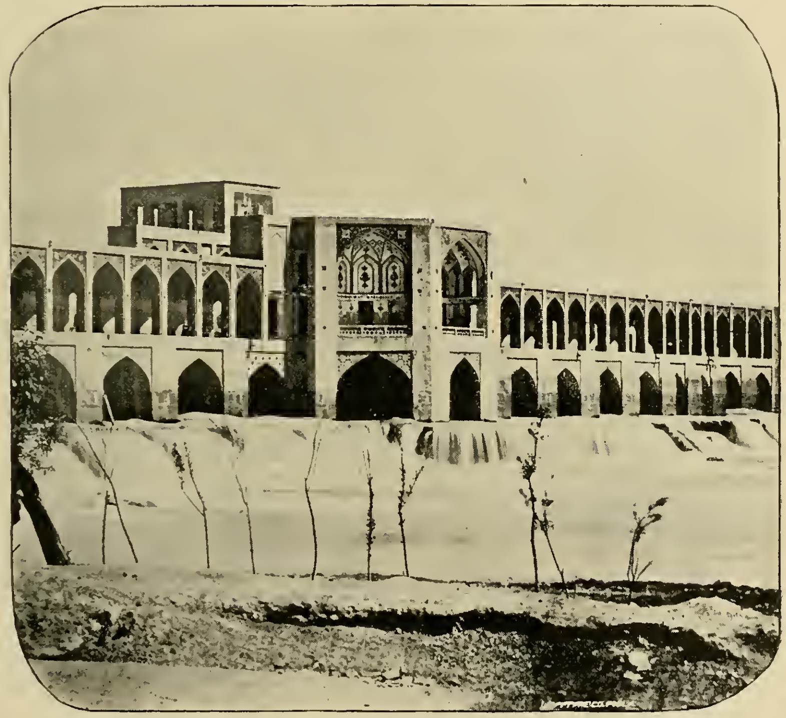


FIG. 15.—BRIDGE AT ISPAHAN, PERSIA.

in 1683 by the Dominican monk Romano, the first dredging machine is said to have been employed.

Fig. 15 shows the bridge of Allah-Verdi-Khan, at Ispahan, which has thirty-three arches of about 18 feet span, and three smaller arches at the end, all pointed, and of true Persian character.

Fig. 16 shows a bridge in China, which, in its steep approaches, reminds us of some of the mediæval bridges of Europe. The dates of these bridges are uncertain.

In the eighteenth century a great impetus was given to the rational

and æsthetic design of bridges by the establishment (in 1716) in France of the Corps des Ponts et Chaussées, or government engineers of bridges and roads. Gabriel was placed at the head of the corps, being the first engineer-in-chief. Most of the bridges of the older period—those built by the Romans and by the "Brothers"—had tumbled down, and many new bridges were needed. From this time development was very rapid. After Gabriel's designs, the bridge of Blois, over the Loire, was built by Pitrou, in 1720, consisting of eleven flattened arches, with

Fig. 16.—Chinese Bridge.

spans increasing from 55 feet at the ends, to 86 feet at the center, the grade of the approaches being as great as in many of the older bridges. In this bridge, for the first time since the old Roman days, the centers were supported entirely at the ends, without any intermediate supports whatever. In the bridge at Saumur, a saw was invented for cutting off piles under water at a considerable depth (16 feet), while in the Westminster Bridge, at London, built in 1738, by a French engineer, Labelye, caissons of the modern kind were used for the first time, being sunk upon a prepared foundation of piles covered by a platform.

In 1760, the École des Ponts et Chaussées, originally started in 1747 as a drawing school, was organized for the training of engineers, and the noted engineer, Perronet, was placed at its head. From that time to this the great majority of the public works of France have been built by its graduates, and it is not too much to say that its establishment gave to France that pre-eminence in engineering which she enjoyed for so many years. Perronet's bridge at Neuilly, across the Seine, consisting of five spans of 128 feet, completed in 1774, and justly considered the masterpiece of its builder, was distinguished by the first use of what are known as cow-horns, the object of which is to facilitate the passage of floods and debris under the arches. Segmental arches, of course, owing to the fact that they are not vertical at the springings, but rise gradually, offer more obstruction to the floods than elliptical or basket-handle arches. For this reason, where used, their springing had thus far generally been placed higher than when other forms were used, often much above ordinary high water. After the introduction of cow-horns, by Perronet, in the Neuilly bridge, however, segmental arches were soon used with the springing at the high water-mark, or even below it. The use of cow-horns, moreover, gave an appearance of lightness, even though the arch ring might be thicker than usual towards the abutments. The Neuilly Bridge was of the basket-handle type, with piers semicircular at each end and 14 feet thick. It was founded on piles. Fig. 17 is the bridge at Bordeaux, with cow-horns.

The Pont de la Concorde, in Paris, begun 1787, has five flat segmental arches, with spans from 83 to 102 feet, supported by very slender piers only 10 feet thick, which would doubtless be unable to stand if one arch should fall. This small thickness facilitates the passage of the water, and gives the lower portion of the bridge an appearance of lightness which, however, is destroyed by the heavy treatment of the tops of the piers and the roadway.

To the eighteenth century belong some of the most noted stone bridges of England.

The old Westminster bridge, at London, completed in 1750, was the first bridge in which caissons were used in founding the piers. A settling of one pier, however, required the taking down and rebuilding of the two adjacent spans. This bridge had thirteen semicircular arches with a maximum span of 72 feet. It stood for about a century, till the demands of traffic led to its being replaced by a cast-iron bridge of seven spans, 85 feet wide.

The old Blackfriars bridge, built by the Scotch engineer, Mylne, from 1760–1769, had nine basket-handle arches, with a maximum span of 100 feet and a rise of 40 feet. It was 995 feet long and 45 feet

wide, and was handsomely decorated with double columns over the piers. The bridge cost £152,840. Owing to settling, however, and also, it is said, to the use of poor stone, it was necessary, in 1833, to repair the bridge at a cost of £105,138. Even then it was not satisfactory, and in 1865 it was replaced by a cast-iron arch bridge of five spans, designed by Joseph Cubitt.

The bridge across the Thames, at Kew, is still in existence, and though its dimensions are but moderate, it was carefully and solidly constructed.

FIG. 17—BRIDGE AT BORDEAUX, FRANCE.

Worthy of mention is the bridge over the Taaf, at Pony-y-Pridd, in Wales. In 1746 a mason, W. Edwards, built an arch bridge here of three spans. After two and a half years it was carried away by a freshet, and being guaranteed for seven years, Edwards had to rebuild it, and did so, using a single arch. It was completed, but the parapets had not been put on, when the load on the haunches threw up the crown and the arch fell. Mr. Edwards consulted Smeaton and rebuilt it as before, in 1748, but with openings in the haunches. The span is 140 feet, and the rise is 35 feet, and the thickness of arch ring at the crown 2.5 feet.

The aqueduct at Alcântara, near Lisbon (Fig. 18), is attributed by Gauthey to Trajan, but in reality the Romans made scarcely the beginning, and it was not until 1731 that the work was really begun. It had been nearly completed at the time of the terrible earthquake of 1775, which destroyed the greater part of the city. The aqueduct, however, was so solidly built that it was little injured, and, moreover, it was somewhat distant from the center of disturbance. This aqueduct contains

FIG. 18.—AQUEDUCT NEAR LISBON, PORTUGAL.

35 arches, the central or largest opening with a pointed arch of 100 feet span and 88 feet rise. The height of the intrados of this span at the crown is 197 feet above the river, and the maximum height of the structure is 230 feet. This is said to be the highest existing arch bridge having but one tier of arches.

The principal progress during the eighteenth century appears to have been in the increased use of elliptical or other flattened arches, the reduction in the ratio of rise to span, the use of segmental arches

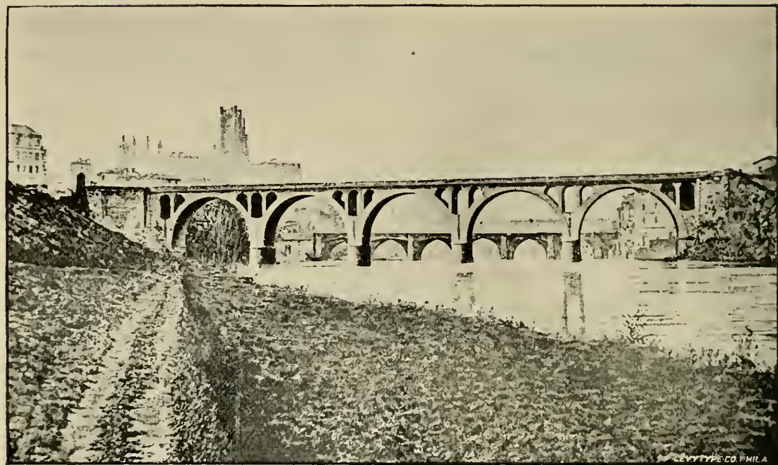


Fig. 19.—Pont Neuf at Albi, France.

Fig. 20.—Bridge over the Nydeck, at Berne, Switzerland.

with cow-horns, the gradually increasing length of span, the use of more slender and graceful piers, the reduction in thickness of the arch ring consequent upon a better knowledge of the mechanics of the arch, and the increased beauty of proportion and adornment. Progress, too, was made in the methods of foundation, by the use of caissons, dredges, mechanical pile drivers and saws for cutting off piles under water.

In the present century, and especially since the birth of the railroad in 1830, a great many stone bridges have been constructed, but the types have not changed, and the development has simply been in the directions just indicated.

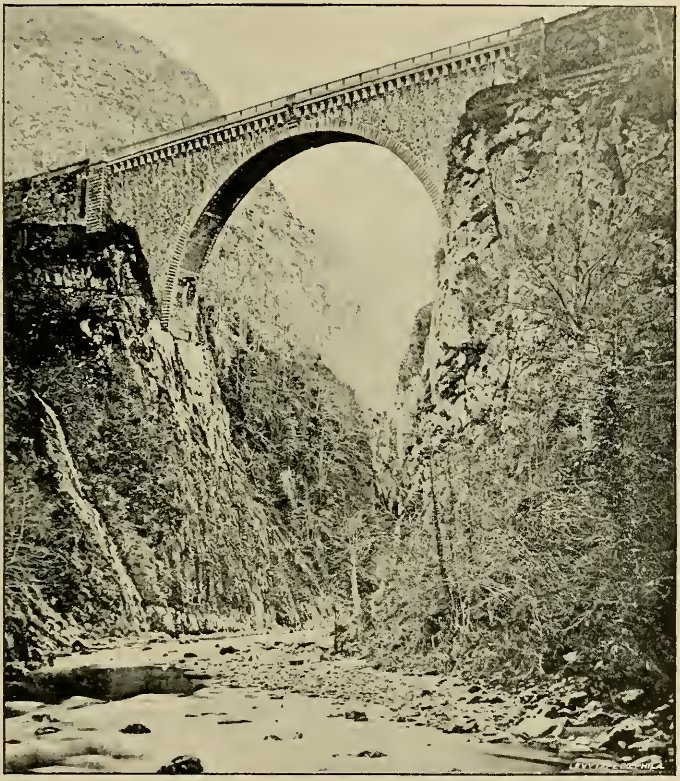


FIG. 21.—BRIDGE OF ST. SAUVEUR.

Figs. 19–20 show some modern bridges. Fig. 19 is the Pont Neuf at Albi, in France, an extremely light and graceful structure. Fig. 20 is the Nydeck bridge at Berne, with a span of about 148 feet, a fine structure. Fig. 21 is the Pont St. Sauveur, in the Maritime Alps, with a

span of 140 feet, and a height of 215 feet above the river. Fig. 22 is a view of two bridges, the older having steep approaches and a small width, to remedy which a modern bridge has been built alongside, nearly level, and of sufficient width for the increased traffic; Fig. 23 is the "Viaduc de Morlaux," and Fig. 24 is the famous Roquefavour aqueduct, in France, the highest stone bridge in the world.

These views show that there is no very great or essential difference between stone bridges of the present day and those of the Romans. There are differences in detail, with regard to the shape of the curve of the arch, the arrangement of the stones, the bonding,

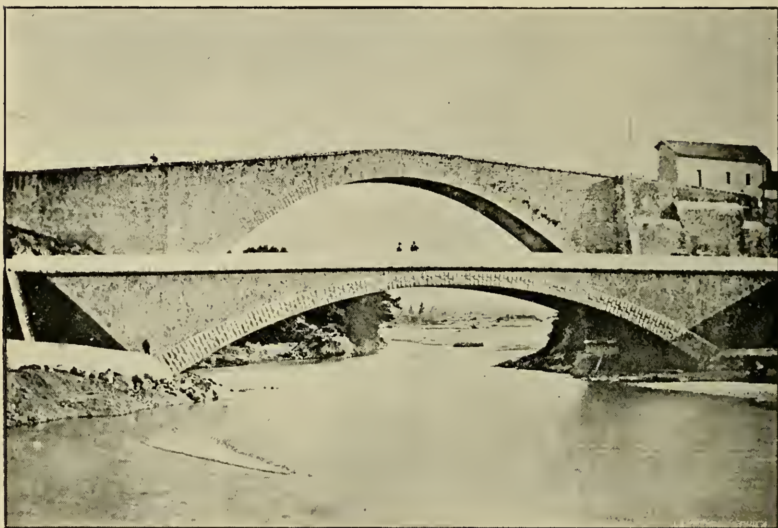


Fig. 22.—Ancient and Modern French Bridges.

the proportions of the piers, etc.; but the general principle and construction remain the same.

In America there are few large stone bridges. This is but natural in a new country. Not strange is it, either, that most of our large stone bridges are on works of water supply rather than on railroads. Our country has been built up by the railroads, which have penetrated in advance of civilization into new regions, and have been too poor at the beginning to build such costly structures—unlike the railroads of Europe, which were built through already rich and populous districts. Aqueducts, on the other hand, are built to supply already wealthy and populous cities, and the most durable and expensive structures are justified.

Of railroad bridges there are but very few large structures. The

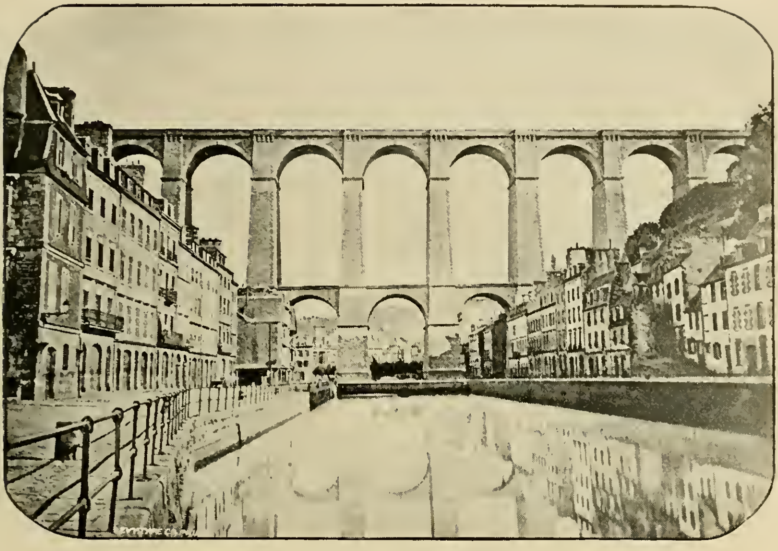


FIG. 23.—VIADUCT OF MORLAUX, FRANCE.

FIG. 24.—ROQUEFAVOUR AQUEDUCT, FRANCE.

FIG. 25.—CABIN JOHN BRIDGE, NEAR WASHINGTON, D.C.

FIG. 26.—ECHO BRIDGE; SUDBURY AQUEDUCT, NEAR BOSTON, MASS.

earliest was the Thomas viaduct, between Baltimore and Washington, over the Patapsco River, with eight elliptical arches on a curve. The spans are 58 feet, and the height above the river 65 feet.

The next is the Starucca viaduct, on the Erie road, in northern Pennsylvania, with eighteen arches of 50 feet span and 110 feet high.

A third is the Canton viaduct, on the Boston & Providence Road.

All of these, however, compared with the structures which have just been described, are unimportant works.

Of aqueducts, the first is High bridge, over the Harlem River, in the old New York aqueduct, with seven spans of 50 feet and eight spans of 80 feet, the height above the water being about 100 feet.

The second is the Cabin John bridge (Fig. 25), over the creek of the same name, in the aqueduct supplying Washington. This was built by Gen. Meigs in 1866, and has one segmental span of 220 feet, with a rise of 57 feet 3 inches, being the largest stone arch span in existence, and only exceeded in the world's history in the case of the bridge at Trezzo, already mentioned. This bridge is but 20 feet wide.

The third is the Echo bridge (Fig. 26), on the Sudbury River aqueduct, supplying Boston. This has a span of 127 feet and a rise of 42 feet.

(The lecture, of which this is a portion, was illustrated by a great many views, which space will not permit us to insert, showing most of the bridges referred to in the text, and many others not specifically described.)

Bradley & Poates, Engr's, N.Y.

JOURNAL OF THE ASSOCIATION OF ENGINEERING SOCIETIES, 1896.

Errata in November number, page 146, lines 21, 22. For June and December read March and September; for March and September read June and December.

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SOLAR WORK IN LAND SURVEYING.

A Statement of the Principles Involved and an Exhibition of a New Solar Device.

BY J. D. VARNEY, MEMBER OF THE CIVIL ENGINEERS' CLUB OF CLEVELAND.

[Read before the Club, October 13, 1896.*]

IT appears that descriptions of land lines by courses and distances, such as we now use, are found in the cuneiform writings of Babylonian relics. The long use of that form is not proof that it is best, but that it is the best is, I believe, the reason that it has been used so long. Whereever another form appears on our records, it impresses us as being an exhibition of pedantry.

It would be of advantage if all our courses could be noted with reference to the true meridian, but the labor required to run them so has heretofore been such that the cost has seemed to outweigh the benefits.

The purpose in making all original surveys should be to so do our work that it can at any later time be reproduced with the least possible labor and the highest degree of accuracy. If monuments are placed to mark every line, there will be no future uncertainty so long as the monuments remain. If increased facilities for finding the true meridian shall tempt surveyors to neglect setting monuments, then by so much will those facilities work harm.

One of our members, Mr. John B. Davis, has devised and patented an instrument for determining the meridian by a method which, he claims, not only reduces the labor but also increases the accuracy of the work.

* Manuscript received November 4, 1896.—*Secretary, Ass'n of Eng. Socs.*

It is the purpose of this paper to introduce to you one of his instruments which was manufactured for me by Messrs. Ulmer & Hoff of this city, and to give a correct idea of its construction requires that I should describe the principles involved.

The earth is revolving around the sun at a distance of about 93,000,000 miles. The plane in which it revolves is called the plane of the ecliptic. The earth is revolving also on its axis, causing the phenomenon of day and night.

This axis or line passing through the center of gravity and around which it revolves daily is not perpendicular to the plane of the ecliptic, but is inclined about $23\frac{1}{2}°$ from the perpendicular. For the purposes here treated, this angle may be regarded as constant, and the position of this axis at any given time, as therefore parallel with its position at any other given time. The equator is an imaginary line passing around the earth midway between the poles. As the axis is necessarily at right angles to the plane of the equator, it follows that the planes of the equator and of the ecliptic are at the same constant angle as the angle of the axis to the perpendicular. From these facts it follows that, as the earth revolves around the sun, there are two positions in which the sun is in both planes, those of the ecliptic and the equator. These occur in June and December. Then the sun appears to travel alternately north and south until, in March and September, its angular distance from the equatorial plane is equal to the angle between the planes of the ecliptic and the equator. This angular distance of the sun from the equatorial plane at any specified time is called the declination of the sun for that time.

The lines of which we as land surveyors treat are not lines, they are planes. The center of the earth is a point in each of them and they extend indefinitely upward. We treat of them as extending far enough vertically to include the points of which we speak as being in line. When we speak of three points as being in line, we mean that the center of the earth is a fourth point in the same plane as the three given points.

The meridian of any point is a plane passing through that point and through the earth's axis. The whole problem in this inquiry is: From a given point in the earth's surface, to find another point in that surface which shall be in the same meridian plane, and to do this by observations of the sun.

If a circle of the size of the earth's circumference could be marked around the center of the sun so that we could use it as a target, it would be about like using a target 1 inch in diameter at a distance of 1000 feet. Whether the line of sight is directed to the side or to the center of such a target makes an angular difference of about 9 seconds. As affecting solar work, this parallax would be produced only at one of the poles of

the earth, and, as we are not likely to go there to do solar work, and as, in all lower latitudes, the angle would be less, we may ignore it and treat all the angles we use as if made at the center of the earth, from which point, I understand, astronomical calculations are usually made.

Let us suppose the earth to be so placed that its axis is perpendicular to, and that its equatorial plane coincides with the plane of the ecliptic, and that our point of observation is on the equator in the "sun-rise" position. By "sun-rise" position, I mean that position in which a plane tangent to the earth's surface will strike the center of the sun, subject of course to a parallax error.

If the conditions are plainly pictured, it will be seen that if with an ordinary transit we should turn 90° from the sun, we would bring the optical axis of the telescope in the meridian plane, and our given problem would be solved.

With a line given, and another line to be determined by turning a given angle from a given point in the given line, there are two methods by which we can turn that angle.

For the sake of brevity I will, for the present, discuss only right-angles, assuming that the modifications for other angles are so obvious as to require no special description.

Having the line *A B* (Fig. 1) given and wishing to find the line *B C* which is known to be perpendicular to *A B* from *B*, the usual method is as follows: Set up and level the ins.rument over *B;* set the zeros; direct the line of sight to *A*, using the lower movement; turn the upper plate 90°, and the line of sight will then necessarily be in the vertical plane *B C.*

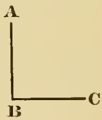


Fig. 1.

The mechanism used in all solar work which will here be considered is such that two lines of sight are so attached that they can be set and clamped at any desired angle to one another. Assume that we have such an instrument placed over *B* with the lines of sight clamped at right-angles to each other. Obviously, if one is in the vertical plane *A B*, the other must be in the vertical plane *B C.*

If, for the lines of sight we use telescopes, if, to one of them, we attach a reflector clamped at 45° to its optical axis, and if we so place the instrument that the point where the optical axis of the telescope strikes the reflector is in the vertical line passing through *B*, the image of *A* can be brought into the optical axis, when, and only when, that optical axis is in the vertical plane *B C.*

Taking advantage of this well-known law of optics, one telescope is made to suffice instead of two. This is the method of the Davis Solar.

We are now to describe Mr. Davis' method of fixing this reflector at right-angles vertically and at the required horizontal angle to the optical axis.

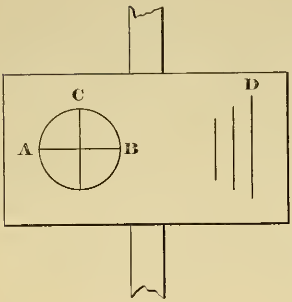


Fig. 2.

A target is required and Fig. 2 represents the form used. The line $A\ B$ is to be made horizontal. The other lines are perpendicular to $A\ B$. D is to be used when the reflector is to be set at 45° for right-angles. The distance between the lines C and D is equal to the distance from the transit axis of the telescope to the axis of the reflector. The remaining lines are for other than right-angles, and are at distances from the line C equal to the sine of the required angle multiplied by the distance $C\ D$.

Assume that we have a transit such as is used in ordinary survey work, with level on telescope, and clamp and tangent movement on the vertical axis. Set it at random. Level the plates and telescope. Set the zeros of the limb. Have the target placed at a convenient distance, with its face at right-angles to the line from the instrument. Have it raised or lowered until the line $A\ B$ is covered by the horizontal hair. With the lower screw fix the vertical hair on D. Turn 90° to the left. In place of the sun-shade, fix this reflector attachment, which consists of a ring fitting around the object glass, close enough to have no lost motion and loose enough to be turned easily. By an upper and lower bar a plane reflector is attached to this ring, a short distance from the object glass. Being supported by pivots, this reflector revolves on its axis at or near its middle line. We revolve it by the ring to make it vertical, and on its pivots to bring it to the required angle—until the vertical hair covers the line C of the target, and the horizontal hair covers the line $A\ B$. When this occurs, we know that the face of the reflector is at right-angles vertically and at 45° horizontally to the optical axis of the telescope.

In this position let us clamp it and take it with us to our assumed "sun-rise" position on the equator.

Do not forget that the plane of the equator is now assumed to coincide with the plane of the ecliptic.

Level all the parts. This being done, the optical axis will revolve in a plane tangent to the equator, when the telescope is revolved by the plates. The sun is in that plane.

By revolving the plates, bring the image of the sun on the cross of the hairs. The optical axis will necessarily be in the meridian plane, and again our problem is solved.

Clamp all the parts in this position and lêave it while we introduce another modification of the engineer's transit, made necessary by the fact that we cannot make all our observations at sunrise.

In place of being rigidly fixed in its transit axis, the telescope is placed in a sleeve in which it can easily be revolved on its optical axis as level telescopes are revolved in their Ys. As it is this sleeve, and its relation to the reflector, that constitute the departure of this from all other devices, it is important that for a time I digress from the discussion of astronomical conditions and speak more fully of this mechanical construction.

The object glass is fixed, hence all focusing is at the eye end of the telescope. The reflector is so attached at the object end that when, by the slow motion screws, it has been placed in the relation to the optical axis required for any specific observation, it can be clamped and held in that relation as if it had been constructively made for that observation and for that only. When the telescope is revolved in the sleeve, the rays of light which are reflected into the optical axis form a plane perpendicular to the optical axis if the angle is 45°. If not, they form the surface of a cone, the apex of which is in the optical axis and at the point where it meets the surface of the reflector.

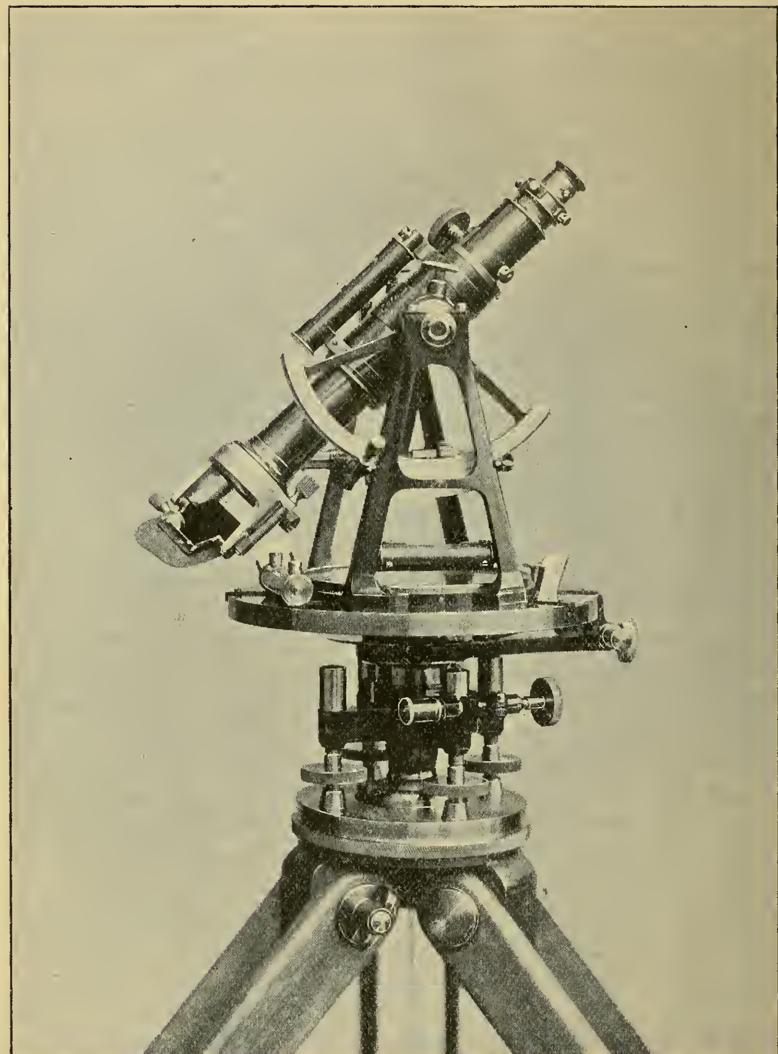
This device is not new in all its parts. Reflectors have been used and they have been attached to a ring on the object end of the telescope to serve the purpose of the hour arc. Usually they have been set by graduated arcs and verniers. Others have a stationary object glass. All exclusively leveling instruments revolve on the optical axis in the Ys. All former devices have, however, required adjustment. The combination made by attaching the reflector to the fixed object end, setting it by means of the graduated limb of the transit, and then, by means of the sleeve, revolving the telescope on its optical axis, and, by means of these, eliminating all adjustments which are not required for other transit work, is the crowning feature of this device.

Setting the reflector by using the target is not an adjustment, but by so setting it, results are obtained which in all former devices have required three or more delicate adjustments. All the usual adjustments of the transit must be carefully made, but that is required if any other work is to be well done.

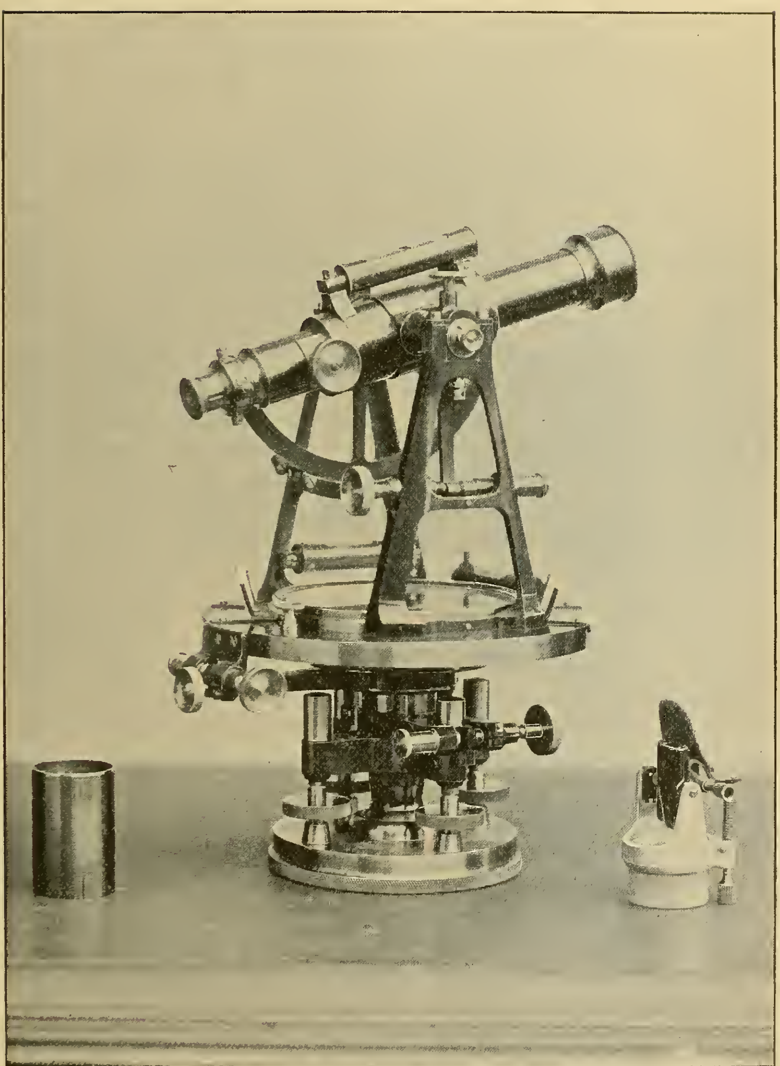
Remove this reflector and we have simply an engineer's transit improved by the fact that the sleeve enables us to adjust the line of colimation by sight in one direction as in Ys.

Going back we now place this sleeve modification in our mental picture of the clamped instrument which we left standing on the equator. The optical axis was left on the meridian plane. The light of the center of the sun was thrown by the reflector on the cross of the hairs.

Keeping the instrument and all its parts stationary with reference to the earth, let us leave it for some definite length of time, say two hours, in which time the earth will have turned on its axis 30°. The



J. B. Davis Solar Transit. Reflector Attached and Instrument Ready for Observation.



J. B. Davis Solar Transit. Reflector Detached.

appearance, to us, will be that the reflector is still directed to the "sun-rise" position, but that the sun is two hours high.

It will, however, be seen that if we had turned the telescope in the sleeve with the same angular velocity with which the earth turned on its axis, that is, 15° per hour, and without changing the optical axis from its position parallel with the earth's axis, the vertical hair would have continued to bisect the image of the sun during the whole movement.

If now we turn the telescope in the sleeve 30°, so as to bring the image of the center of the sun on the cross of the hairs, the optical axis of the telescope will have remained in the meridian plane, but the vertical hair will stand at 30° to that plane. If, again, we wish to mark a point in that plane, we have only to turn back the telescope in the sleeve until the vertical hair is vertical, remove the reflector and proceed in the usual way. So much for the method in the "sun-rise" position.

Let us now commence an observation on the equator at some other time than sunrise, the assumption continuing that the planes of the equator and the ecliptic coincide. So far as concerns this observation, the time may be unknown, except that, for reasons to be hereafter given, it must not be at noon and for the most reliable work it should be between 8.00 and 10.00 A.M. or between 2.00 and 4.00 P.M.

Please do not forget the assumption that the plane of the equator coincides with the plane of the ecliptic.

Level all parts of the instrument. Using the target in the manner before described, set the reflector at the required angle, 90°. Revolve the telescope horizontally by one of the plates and on its optical axis in the sleeve until the image of the sun is bisected by the vertical hair. If the conditions are clearly pictured, it will be seen that this can only occur when the optical axis of the telescope is in the meridian plane, but the vertical hair will be at an angle to the vertical or meridian plane equal to the angular distance of the sun from the "sun rise" position. To mark out the meridian, we must turn the telescope in the sleeve until the vertical hair is vertical, remove the reflector and proceed in the usual way to mark the line, and again our problem is solved.

Because some mental effort on my part was required to see why solar observations near noon are unreliable, and at noon, valueless, I may be pardoned for giving some time to that subject. Keep in mind our position on the equator, the plane of which is assumed to coincide with the plane of the ecliptic; also keep in mind that the lines under discussion are not simply lines, but sections of planes.

The name noon is given to the time when the sun is in our meridian plane. Hence to direct the telescope to the sun at noon we must place the optical axis vertical. In that position no angle can be measured

because to revolve the upper plate is only to revolve the optical axis on itself.

To illustrate further. Place the instrument over *B* (Fig. 1), to observe the angle *A B C*, which, for this purpose, may be more or less than 90°. Turning from *A*, we usually look, not at the actual monument *C*, but at a flag-pole placed in the vertical plane *B C*. Let us assume this flag-pole to be in the form of a curve, the center of which is at the instrument. From *B* we may observe the angle *A B C*, turning from *A* to successive points on the flag-pole above the horizontal, but as we approach the perpendicular our work will be less and less reliable, until at the perpendicular, we simply turn the telescope on its optical axis. When the telescope is elevated 60° from the horizontal, or 30° from the vertical, the conditions are practically the same as those of solar work at 10.00 A.M. or 2.00 P.M.

Let us now inquire as to the modifications necessitated by the fact that the planes of the equator and the ecliptic do not coincide. To simplify our conception, let us eliminate the reflector and assume that we turn angles in the usual way. Let us go back to our assumed "sun-rise" position on the equator, the plane of which is still assumed to coincide with that of the ecliptic. By leveling up, we make the plates and the telescope tangent to the earth. Bisect the image of the sun with the vertical hair. Taking parallax into account, the horizontal hair will be above or west of the center of the sun a distance equal to the semi-diameter of the earth. Picture a line passing through the centers of the sun and earth and indefinitely extended. Coming from the sun it would strike the earth 90° east of us and pass out 90° west from us. On this line as an axis let the earth revolve. This being clearly pictured, it will be seen that this revolution might be continued indefinitely and that the cross hairs would appear to remain fixed on the center of the sun.

Taking parallax into account, and imagining the optical axis of the telescope to be a pencil extending to the sun, and the sun to be a sheet of paper, the pencil, as the earth revolved, would describe around the center of the sun a circle of the size of the earth, and at all times the vertical hair would cover the center of the sun.

By our terms, adopted at the outset, we have placed the telescope at right angles to the meridiam by directing it to the sun. By our terms the position of the telescope remains fixed with reference to the earth. Therefore, at any point in the supposed revolution, we should find the meridian by turning 90° from the sun. In other words, at that time in March and September when the sun is in the equatorial plane, we should find the meridian by turning 90° from the sun.

Let us again return to our assumed "sun-rise" position, with the planes of the equator and the ecliptic coinciding, with all parts of the

instrument leveled and with the telescope directed to the center of the sun.

Imagine a line passing through the instrument and through the center of the earth and indefinitely extended. On this line as an axis let the earth revolve, the north pole moving toward the sun an angular distance equal to that between the planes of the equator and the ecliptic. By that movement we should have, so far as this inquiry is concerned, the same effects as are produced by the movements of the earth around the sun from March to June. By reversing the direction we should have the effects witnessed from June to December.

Let us now go back and assume the north pole to have moved some definite distance, say 10°, toward the sun. If this is clearly pictured, it will be seen that the line of sight will have left the sun and will now be directed 10° south of it. The meridian will, by our terms, still be at right angles to the telescope. We would, therefore, turn 90° minus 10° (= 80°) from the sun for a north line, or 90° plus 10° (= 100°) from it for a south line.

This angle, taken here as 10°, being the declination of the sun, its value for any hour or minute of each year can be easily determined from data given us by the astronomers, and it is with reference to this only that the time in the day or even in the year becomes a factor in solar work. I make this statement because of the possibility that some others may have the same erroneous conception that I had until recently. Because I was told that a man, engaged in solar work, must carry a watch to know the time of day, I assumed that it was in some way, I did not know how, connected with the 15° per hour that the earth turns on its axis. This would require an accurate timepiece, whereas a fairly good " Waterbury " will do.

Thus far, our observations have been made on the equator. But we are actually in latitude about 41° 30′ north. Our plumb bobs point to the center of the earth and are not perpendicular to its axis. Our level bubbles, keeping as far as possible from coinciding with the plumb lines, assume positions at right angles to them.

If we could but have such a modification of the laws of gravity that the plumb lines would hang at right angles to the earth's axis, instead of tending toward the center of the earth and that the level bubbles would continue to assume positions at right angles to the plumb line, our work would be simplified and we could do our work on any part of the earth's surface in the same manner as at the equator. As it would be difficult to pass such an amendment to the laws of gravitation, we may as well submit and make the necessary amendments to our transit. This is very simple, after all, requiring only the addition of a vertical arc, with which we are all familiar, to the axis of the telescope.

Knowing the latitude, we know also the angle between the axis of the earth and a horizontal plane. We find a horizontal plane by leveling the plates and the telescope. We elevate the object end of the telescope if we are to look north, or depress it if we are to look south, until the vernier of the vertical arc indicates the correct angle. We then proceed, in all other respects, as if we were on the equator.

It has thus far been assumed that the latitude is known. It now remains to *find* the latitude when it is *not* known. This must be done with as high a degree of accuracy as possible, because errors here are repeated and sometimes multiplied in our results. One thousand feet north or south makes a difference in latitude of between nine and ten seconds.

We must know the declination of the sun at noon of the day on which we are to find the latitude. As this is learned by other means than the use of this instrument, the knowledge of it is assumed. For example let us assume it to be 10° north. It is more convenient to depress the object end than the eye end of the telescope, this instrument is therefore constructed to look south, and the angle we are to deal with is the same as that which we should use if the latitude were known and we were about to find the meridian, that is, $90° + 10° = 100°$.

We go back and follow the instructions for fixing the reflector in position, except that in place of turning off 90°, as then described, we now turn off 100°. This work should be completed some time before noon.

That the plates and telescope are made level is provided for in fixing the reflector. We now set the zeros of the vertical arc and clamp it to the telescope while that is yet horizontal. Loosen the vertical clamp of the telescope. Loosen one of the plates; whether the lower or upper one, is not important. Turn the object end southerly. Turn the telescope in the sleeve so that the vertical hair will be horizontal. Revolve the telescope by the plates and elevate or depress the object end until the vertical hair, now made horizontal, bisects the image of the sun. Keep it there, following the sun by using the slow motion screws when necessary until the greatest elevation of the sun is reached. Before it recedes, read off the angle indicated on the vertical arc. This will be the latitude of the place.

The optical axis of the telescope will then be near the meridian, but we must not assume that it is in the meridian. If in the meridian, it would be parallel to the earth's axis.

I have spoken of the "sunrise" position, assuming that the modifications necessary in applying the discussion to sunset are too obvious to need statement.

I have said that we cannot make all of our observations at sunrise.

In fact, on account of refraction, it is not practicable to make any of them at either sunrise or sunset. Your knowledge of the nature and extent of this factor is assumed.

Accompanying the tables of declination, the astronomers have given us data for correction for refraction.

Between refraction near sunrise and sunset, and the heretofore mentioned inaccuracy near noon, the time favorable for observations is limited to a few hours daily. It has been found that the best results are obtained between 8.00 and 10.00 A.M., and 2.00 and 4.00 P.M.

My efforts for brevity necessitate some further explanations. We do not attempt to bisect the image of the sun with a single hair, but bring its image under two hairs which cover a space a little less than its diameter.

The reflector is made vertical, not actually by turning it on the object glass as intimated, but by a better device, with which, however, it would have been difficult to describe the principles involved.

I have spoken of removing the reflector to mark the points in the meridian. This is not necessary, because, if it is so turned that it is in or near the line of the optical axis, it ceases to be an obstruction to vision. It can, however, be easily removed, and is kept on the telescope only when solar work is to be performed.

As in other astronomical work, an inverting eye-piece is used. This is not a necessity, but by its use mechanical difficulties are more easily overcome and, personally, I prefer it for ordinary transit work.

By a simple though not easily described device, the revolution of the telescope in the sleeve is arrested when the vertical hair is vertical.

A shaded glass is used at the eye-end to enable us to look at the bright sun.

Persons familiar with other solar instruments will observe that the limb of the ordinary transit is here used as the declination arc.

Since this instrument was finished there has been but one day available for solar work, and on that day there was available time only for two observations. One was made by Mr. Davis, and the other by an observer who was unfamiliar with the work. On a meridian previously established, one struck 15 seconds east and the other 15 seconds west.

Mr. Davis, who has used (since July, 1895) the first instrument made, says that his maximum error is 30 seconds.

In conclusion, I feel that it is no more than is due to Messrs. Ulmer & Hoff that I be allowed to say that the workmanship of the entire instrument is as near perfection as that of any instrument I ever examined. More than this, an examination will show that they have made minor improvements to the instrument as a transit, which show that every step in its construction has been thoroughly, conscientiously, and scientifically thought out.

BOILER EFFICIENCY, CAPACITY, AND SMOKELESSNESS, WITH LOW-GRADE FUELS.

By William H. Bryan, Member of the Engineers' Club of St. Louis.

[Read before the Club, October 21, 1896.*]

Probably the most interesting question now before the mechanical engineers of this country is the proper method of expressing the efficiency, or economy, of steam boilers, and with this is closely connected the entire subject of the methods of making evaporative trials.

Active discussion of the matter was precipitated at the Detroit meeting of the American Society of Mechanical Engineers in 1895, when a paper by Mr. F. W. Dean, of Boston, on "The Efficiency of Boilers, a Criticism of the Society's Standard Code of Reporting Boiler Trials," was read. Mr. Dean not only recommended a complete revision of this code, but took strong ground in favor of determining and reporting, in every boiler trial, the heat value of the fuel used, and comparing that figure with the actual heat utilized in doing useful work in the boilers.

The discussion which followed the reading of this paper was interesting and extensive, and it continues unabated to the present day, as is indicated by papers presented at both meetings of the Society since that date and by correspondence in the technical journals. As a result of Mr. Dean's paper, the Council of the American Society of Mechanical Engineers appointed a committee to consider the standard method of 1886 of conducting steam boiler trials, and to recommend to the Society such revision of that standard as might seem desirable to them. The committee appointed consists of Messrs. Geo. H. Barrus, J. S. Coon, F. W. Dean, Chas. E. Emery, Wm. Kent, R. W. Hunt, C. T. Porter, Prof. W. B. Potter of this city, and Dr. R. H. Thurston. This committee is now engaged on the work entrusted to it, but it having proved to be an undertaking of great magnitude, involving the consideration of many intricate problems, there is no telling when their report will be presented to the Society.

It is generally conceded that the Society's code needs revision. At the time of its preparation it represented the best practice, and the fact that it has stood until now without material modification, or even serious criticism, is the best compliment that can be paid its authors. At that time, however, boiler trials were only rarely made, and the

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methods and apparatus had been but imperfectly developed. In fact, the entire theory of boiler performance was but imperfectly understood. Since then the making of boiler trials has become general, many thousands of them having been made, and they are now matters of almost daily occurrence, particularly in the larger cities. It is not surprising therefore, that the great development of this branch of engineering, should have brought to light some features which were not considered at the time the present code was prepared. For these reasons the code should now be so modified as to bring it into harmony with the best practice of the present day.

Perhaps the most important question involved in this revision is as to the best method of expressing the economic performance of a boiler. Broadly speaking, boiler efficiency means the ratio between the work done by the boiler and the fuel expended per unit of such work; or, to put it more precisely, the ratio between the heat realized in useful work by the boiler and that existing in the coal which the owner has paid for.

Until recently the economic performance of a boiler has always been expressed in the number of pounds of water evaporated for each pound of fuel burned, and for each pound combustible; the latter term meaning simply the remainder after deducting from the total coal burned the pounds of ash weighed back. There being wide variations between the temperatures of the feed water, and the pressures carried at different trials, the above results were always reduced to the "equivalent evaporation from and at 212," which is what the boiler would have done had the temperature of the feed water been 212°, and the water evaporated at the same temperature, or at atmospheric pressure. This enables comparisons to be made between different trials under different conditions of feed and pressures.

For some years past many engineers have believed that the efficiency thus stated did not tell the whole story. The "equivalent evaporation per pound of coal" is meaningless, unless we know all about the coal. The situation is not much improved by reducing it to the "pound combustible," as this latter term is easily liable to error, particularly with low-grade fuels running high in ash, and making bad clinkers, such as are almost universally employed in this part of the country. These engineers strongly urged that in every boiler trial the coal burned be carefully sampled, and immediately submitted to chemical analysis and calorific determination. The report of results should show these facts as well as the theoretical evaporative power or capacity of a pound of coal. Comparing this with the work actually done by a pound of the coal, the efficiency percentage is at once secured. In the opinion of many engineers, this figure is, everything considered, much the best method of expressing the economic performance of a boiler.

Much to our surprise, the agitation of this subject has developed remonstrances and from quite unexpected sources. Some of the most noted engineers of this country, supposedly most familiar with boiler trials, have taken the position that the efficiency thus determined and stated would in no way improve the situation, and might, indeed, be misleading. These engineers do not seriously defend the "pound combustible"—in fact, they practically admit all that has been urged against it. Their sole ground of objection is the belief that the science of coal calorimetry has not yet reached such a point that implicit confidence can be placed in the results.

Eastern engineers, whose work is almost wholly with high-grade coals running very low in ash, have not experienced the trouble with the "pound combustible" that we have. Very few Western engineers share with them the lack of confidence in calorimeters. My own files contain more than one hundred such determinations, made with all sorts of fuels, under all sorts of conditions, among them many different investigations of the same fuel at different times. The results show a consistency which cannot help inspiring confidence in the methods. Even if we admit that the coal calorimeters are not all they might be, the extended experiments which are now being made in this direction, and the resulting constant improvement in apparatus, must certainly bring about in the very near future—if they have not already done so—methods and apparatus which will meet every requirement.

While I am a hearty believer in the efficiency, as determined by the more modern methods, I believe nevertheless that the results per pound of coal and of combustible—both actual and equivalent—should be given in all reports, if for no other reasons than that everybody expects them, and that a report would hardly be considered complete without them. I am always very careful to state in addition, however, the efficiency in percentage realized of the calorific value of the fuel. It is then entirely a matter of choice as to which of these expressions the individual reader will give the most weight.

The modern steam boiler is a creditable piece of engineering. Its uses are almost without limit, the most important being the heating of buildings, the generation of steam for steam engines, cooking, boiling and making distilled water for ice manufacture, etc.

The pressures carried regularly vary almost as much, being from 0 to 10 pounds per square inch for heating, from 70 to 100 for ordinary service, 125 to 150 for compound or compound condensing engines, and from 175 to 200 on steamboats and locomotives, and where high-duty triple or quadruple expansion condensing engines are employed.

The performance of a boiler may be measured in many different ways:

A. By the capacity developed ordinarily and as a maximum.

B. By its fuel economy, under all the varying conditions of service.

C. By the dryness of the steam which it generates.

D. By the smokelessness of its performance, a condition which is of decided importance in large cities, and which depends as much on the particular setting or furnace employed as upon the boiler itself, if not more.

Evaporative trials are made to determine all the above features. They are also made to secure information on any of the following questions:

A. The value of the different fuels, to ascertain what grade of fuel will evaporate the most water for a dollar. In such trials all conditions must be kept as nearly constant as possible.

B. To determine the effect of different fuels, not only on the cost of doing the work, but also on the capacity, durability and smokelessness of the plant and on the labor required to handle it.

C. To determine the relative efficiencies of different types of boilers, particularly of new and untried designs.

D. To secure the same information regarding different furnaces, stokers or grates, all as applied to the same boiler; it being necessary to determine not only the difference of economy of the fuel, but also the effect of the changed conditions on the capacity, labor, smokelessness and durability of the plant.

E. To determine the effect of different conditions of operation, such as greater or less draft, mechanical draft, either forced or induced; different methods of firing; different conditions of the boiler as to cleanliness; different rates of evaporation, to determine the point at which the fuel economy is a maximum, and the extent to which the boiler can safely be crowded in emergencies.

The most common necessity for boiler trials arises from boiler contracts which include specific guarantees of performance. When boiler or furnace manufacturers make large claims for economy, it is natural that the purchaser should embody those claims in the contract, and require a demonstration of them before accepting and paying for the plant.

Scientific and accurate determination of boiler performance has of late years received an impetus from the growing practice of paying for boilers on the basis of their economic performance. Every additional pound of water evaporated per pound of fuel burned, means a noteworthy saving in annual operating expenses, which saving can be reduced to dollars and capitalized, thus furnishing a measure of the increased investment, which the purchaser can afford to make in the first cost of a boiler of more economical type. When this plan is

followed, bids for boilers are asked for on some assumed basis of efficiency, frequently 65 per cent. to 70 per cent. After the boilers are ready for service, they are to be tested thoroughly by disinterested experts, and the contractor is paid a bonus on each boiler for every 1 per cent. which is realized over and above the stipulated basis. In case the results fail to reach the basis, then the contractor suffers a deduction for each 1 per cent., the amount being either the same as for the bonus, or greater.

When bidders make up their proposals in a case of this kind, they are supposed to know exactly what efficiency their boilers will develop under the conditions set forth in the specifications; or, if they do not know, they must ascertain. Having completed their estimate of the cost of the boiler, and decided upon a figure which they must realize for the installation, they either add to or deduct from that figure a sum depending upon the efficiency which can be secured. Suppose, for instance, a builder desires to realize $3,000 per boiler on a proposed installation, on which the efficiency demanded is 70 per cent., and a bonus of $250 is to be paid for each 1 per cent. above that point, and the same amount deducted if the basis is not reached. If the bidder is certain he can reach 75 per cent. he would be sure of a bonus of $1,250 on each boiler, and could safely reduce his bid by that amount, offering the boilers for $1,750 each.

The recent bidding for the new boilers of the Baden Pumping Station of this city is a prominent example of this method of procedure, the bids being about one-third lower than the same boilers could usually be purchased for in open market, the contractors nevertheless expecting in the end to realize the full selling price, or more. This method makes it to the contractor's interest to build a plant which represents the very highest improvements in economic performance, and should insure to the purchaser a plant which can be operated for the minimum expenditure for fuel.

The following table is an abstract of the results of a number of boiler trials made by the writer on the conditions and fuels common in this vicinity, nearly all of them being made on ordinary Illinois fuels. They represent the widest extremes of practice, including many badly designed and overworked boilers:

Kind of Boilers.	Number of Trials.	EFFICIENCY—PER CENT. Maximum.	Minimum.	Average.
Small vertical	3	46.10	34.60	41.60
Large "	3	52.30	49.00	50.90
" improved setting . .	1	. . . . . .	One trial only	67.89
Tubular boilers	14	60.17	44.76	51.53
" improved setting	34	76.38	41.94	58.87
Water tubular	13	70.11	49.37	61.31
" improved setting .	18	81.32	49.30	67.52
		SMOKE EMITTED—PER CENT.		
Common furnaces	7	75.42	11.09	46.52
Improved "	40	43.40	.29	9.45

The figures are quite interesting, indicating, among other things, that the best improved furnaces increase the efficiency over 25 per cent. above the best common setting under a tubular boiler, and about 15 per cent. under a water tubular boiler. The average daily results of the plants taken just as we find them, show that the improved furnace improves the efficiency of the tubular boiler about 15 per cent. and that of the water tubular about 10 per cent. The best water tubular boiler does over 15 per cent. better than the best tubular with ordinary setting, the increase, however, under average conditions being about 20 per cent. It is interesting to note that the *minimum* efficiency secured, both with tubular and water tube boilers, is lower with alleged improved settings than without them.

Scarcely less interesting are the results obtained regarding the prevention of smoke. The data from common furnaces represent them just as they were being operated in regular service. Those from the improved settings represent good, bad and indifferent devices. They show that the maximum smoke with improved devices is but little over half that from the common furnace; while the minimum is reduced to an imperceptible figure. As an average these figures indicate that improved furnaces have reduced the smoke fully 80 per cent. If we were to eliminate from this list all the notably poor devices, the average of the rest would show a reduction of from 90 to 95 per cent. Of forty

trials of improved, or so-called "smokeless" furnaces, during which careful records were kept of the smoke,

3	average	less than half of			1	per cent.
6	"	between	½	and	1	"
11	"	"	1	"	5	"
7	"	"	5	"	10	"
7	"	"	10	"	20	"
6	"	above			20	"

Hence it appears that smoke averaging less than 1 per cent. is not only possible, but is being secured regularly in every-day service in quite a number of large steam plants in this vicinity.

On page 165 will be found details of an interesting series of trials made by the writer some time ago at the Green Tree Brewery in this city. The boiler plant consisted of four ordinary horizontal return tubular boilers, each 72 inches diameter, 20 feet long, and having 68—4-inch tubes. The boilers are set in two batteries of two each, each pair of boilers having a single furnace. The setting is the Hawley down draft, which is so common in this city as to need no description at this time. The writer discussed down-draft furnaces in general at the Detroit Meeting of the American Society of Mechanical Engineers in 1895. See Transactions of that society, Vol. xvi, page 773.

In the writer's opinion, this Green Tree Brewery plant represents excellent practice, and is admirably designed to secure, in one plant, the maxima of efficiency, capacity and smokelessness. On the first day we ran the boilers at their rating, with damper partly closed; the second day the run was made with damper wide open, but without slicing or disturbing the fires; the third run was made with damper wide open and the fires sliced and agitated, with a view of getting the maximum capacity out of the plant. The results are very interesting. It will be seen that the capacity increased from rating to 81 per cent. and 120 per cent. above rating, while the efficiency dropped from 76.38 to 70.33 and 68.83, the smoke in all three instances being less than 1 per cent. As a whole, this performance has never been equaled, so far as I know, with ordinary tubular boilers, and is rarely exceeded even by the best types of water tube. It demonstrates conclusively that smoke abatement is not inconsistent with the highest fuel economy and the maximum demands for capacity.

On page 167 will be found a summary of fuel analyses and calorific determinations made in connection with some of the writer's boiler trials. They indicate clearly the class of fuel which is commonly met with in this part of the country, and fairly represent the range covered. The high percentages of volatile matter and of ash and sulphur are particularly noticeable. In spite of these drawbacks in the fuel, recent trials of boiler efficiency show that with properly designed boilers

and furnaces, intelligently handled, it is possible to reach practically as high results relatively with these coals as with the higher grades common throughout the Eastern States.

In designing a boiler plant to give the best results under all conditions of service, with low-grade fuels, the following desirable features should be kept in mind:

A. Ample draft; 1 inch of water or even more. Good results cannot be secured with drafts less than ½ inch. Good draft and thick beds of fuel permit the high fire-box temperatures which we have found absolutely necessary.

B. Large ratio of heating to grate surface, so that while burning coal at a high rate per square foot of grate per hour, there is sufficient heating surface to reduce the temperature of the flue gases to 450° F. or less.

C. The combustion chamber should, if possible, be separate from the heating surfaces, so as to avoid their cooling effect. It should be quite deep—30 inches or more.

To secure the very highest results, the gases, after leaving the boiler-heating surfaces at not exceeding 500°, should be passed through feed-water economizers and thence through air heaters. The feed water, leaving the ordinary exhaust heater at a littleabove 200° F., may be raised to over 300° in the economizer, and the heated gases reduced to 250° or less. This reduction in temperature, of course, destroys the usefulness of these gases as draft producers, unless the chimney is very tall. The draft, however, can be better produced by exhaust fans, which draw the air through and out of the furnace and economizer, and discharge the gases at such a height above the roof that they will not be objectionable, thus doing away entirely with the necessity for high chimneys. Still better economy may be secured by placing air heaters in the smoke flue, beyond the fan, or between it and the economizer. Through these the air, entering the ash pit for purposes of combustion, may be drawn, so that the heated gases are finally discharged at a temperature but little above that of the atmosphere. The speed of the fan may be controlled by an automatic regulator, which increases the speed of the fan engine as the steam pressure drops, and reduces it as the pressure increases, thus performing all the functions of an automatic damper regulator. This plan is not experimental or untried, but has already been adopted in numerous large plants.

From the above it is evident that while the best modern water tubular plant, with improved setting, shows an efficiency 60 per cent. greater than the average efficiency of the common tubular boiler and setting, found in this vicinity almost exclusively until a few years ago, there is still room for considerable saving. Whether all these refine-

ments will pay in any given case, can be determined only by consideration of all the conditions. With fuel as cheap as it is in St. Louis, there is, of course, a limit to the amount of money we should spend in improving a plant with a view of economizing fuel bills. As a general rule, however, it may be safely stated that in this year of our Lord 1896, the building of tall chimneys to secure draft, simply advertises the owner's lack of familiarity with modern improvements, or his want of confidence in results easily demonstrated. To this rule there are, of course, exceptions, as for instance, where the plant is small, fuel comparatively inexpensive and money not available, or where other considerations require that the gases be discharged at considerable elevation.

RESULTS OF EVAPORATIVE TRIALS MADE BY WILLIAM H. BRYAN, CONSULTING ENGINEER, ST. LOUIS, MO., ON ONE BATTERY OF HORIZONTAL TUBULAR BOILERS WITH HAWLEY FURNACE, AT GREEN TREE BREWERY, FOR JOHN O'BRIEN BOILER WORKS CO., TO DETERMINE EFFICIENCY, CAPACITY AND SMOKELESSNESS.

Number of Trial	1	2	3
Date 1895	Nov. 19.	Nov. 20.	Nov. 25.
Duration,. . . . . hours	10	8	8
Number of Boilers in Operation...	2 in Battery.	2 in Battery.	2 in Battery.
State of the Weather	Cloudy, High Winds.	Cloudy (Raining).	Cloudy (Raining).
DIMENSIONS AND PROPORTIONS.			
Kind of Boiler	Horizontal Tubular.	Horizontal Tubular.	Horizontal Tubular.
Dimensions of Shell, Diameter and Length.	72 ins. x 20 ft.	72 ins. x 20 ft.	72 ins. x 20 ft.
Number and Diameter of Tubes...	68—4 ins.	68—4 ins.	68—4 ins.
Grate Surface 13 ft. 10 ins. wide, 4½ ft. long . . . . Area, sq. ft.	62.24	62.24	62.24
Water Heating Surface . . . sq. ft.	3,385.35	3,385.35	3,385.35
Superheating Surface . . . sq. ft.	None.	None.	None.
Percentage of Air Space in Grate per cent.	47.3	47.3	47 3
Ratio of Grate Surface to Water Heating surface . . . . one to	54.39	54.39	54.39
Mean Opening of Damper (percentage of full opening). . . .	37 2	97.0	98
Chimney Dimensions, Height and Diameter.	173 ft. x 54 ins.	173 ft. x 54 ins.	173 ft. x 54 ins.
AVERAGE PRESSURES.			
Atmosphere, as per Barometer inches	29.44	29.39	29.225
Steam in Boiler, by Gauge . . lbs. .	95.86	96.17	95.57
" " Absolute . . lbs..	110.56	110.87	110.27
Draught Suction, inches of water . .	0.6417	1.107	1.05
AVERAGE TEMPERATURES.			
Of External Air . . . . . . deg. F.	36.60	24.2	34.0
Of Boiler Room . . . . deg. F.	57.37	54.8	58.44
Of Escaping Gases entering Chimney. . . . . deg. F.	438.9	584.7	564.9
Of Feed Water entering Boiler deg. F.	71.16	53.123	50.2
Of Steam in Boiler . . . . . deg. F.	334.8	337.2	334.7
FUEL.			
Kind of Coal..	Mount Olive.	Mount Olive.	Mount Olive.
Size of Coal	Lump.	Lump.	Lump.
Calorific Power by Calorimeter British Thermal Units . per lb.	10,965	11,024	10,980
Theoretical Evavaporative Power from and at 212° F., in lbs. Water per lb. coal	11.35	11.412	11.367
Total Quantity Consumed . . lbs.	11,250	17,604	21,750
Total Ash, Clinkers and Unburned Coal . . . . .lbs.	730	1554.5	2359.5

Proportion of Ash, etc., to Coal per cent.	6.489	8.83	10.848
Total Combustible Burned. . . lbs.	10,520	16,049.5	19,390.5
Mean Thickness of Fire . . inches	7½	9	10
COMBUSTION PER HOUR.			
Coal actually consumed . . . . lbs.	1,125	2200.5	2718.75
Combustible act'ly consumed, lbs.	1,052	2006.2	2423.81
Per square foot Grate Surface, Coal . . . . lbs.	18.075	35.35	43.68
Per square foot Grate Surface, Combustible . . . . .lbs.	16.902	32.23	38 94
Per square foot Heating Surface, Coal. . . . . lbs.	0.332	0.650	0.803
Per square foot Heating Surface Combustible . . . . lbs.	0.318	0.595	0.716
CALORIMETRIC TESTS.			
Quality of the Steam (dry steam = 100)	98.6796	99.353	99.201
Amount of Water entrained in the Steam . . . . per cent.	1 3204	0.647	0.799
Amount of Superheating. . deg. F.	None.	None.	None.
WATER.			
Amount apparently evaporated lbs.	83,369.34	118,159	142,910
Amount actually evaporated (corrected for entrainment) . .lbs.	82,268	117,295.16	141,782
Factor of Evaporation	1.1856	1.2045	1.2072
Equivalent Evaporation into dry steam from and at 212° F...lbs.	97,536.118	141,282.02	171,163.77
ECONOMIC EVAPORATION.			
Per Pound of Coal:			
Water actually evaporated (corrected for entrainment) . . lbs.	7.313	6.663	6.519
Equivalent from and at 212° F., lbs.	8.669	8.026	7.824
Per pound of Combustible. Water actually evaporated (corrected for entrainment) . . . . . lbs.	7.820	7.308	7.312
Equivalent from and at 212° F., lbs.	9.2715	8.80	8.827
EVAPORATION PER HOUR.			
Water actually evaporated (corrected for entrainment). . lbs.	8,226.8	14,661.89	17,722.8
Equivalent from and at 212° F., lbs.	9,753.6	17,660.25	21,395.47
Per square foot Heating Surface. Water actually evaporated (corrected for entrainment) . lbs,	2.43	4.33	5.235
Equivalent from and at 212° F., lbs.	2.88	5.217	6.320
Per square foot Grate Surface. Water actually evaporated (corrected) . . . . lbs.	132.18	235.57	284.75
Equivalent from and at 212° F., lbs.	156.71	283.74	343.76
EFFICIENCY.			
Percentage of Total Calorific Power utilized, or Efficiency per cent.	76.38	70.33	68.83
Coal Consumed per Horse-power per hour . . . . lbs.	3 98	4.30	4.384
HORSE-POWER.			
Actually developed on basis of 34½ lbs. water evaporated per hour from and at2 12° F.,horsepower	282.71	511.89	620.16
Commercial Rating, at 12 sq. ft. heating surface. . horse-power	282.11	282.11	282.11
Proportion capacity developed is of Commercial Rating . per cent.	100.2	181.09	219.83
Heating Surface required to develop one Horse-power . sq. ft.	11.97	6.613	5.46
SMOKE RECORD.			
Mean Smoke Production on a scale of 100	.700	0.344	0.98

COAL ANALYSIS (AVERAGE).

Moisture	9.27
Volatile Matter	30.87
Fixed carbon	42.66
Sulphur	5.32
Ash	11.88
	100.00

SUMMARY OF ANALYSES AND CALORIFIC DETERMINATIONS OF WESTERN FUELS.

PRINCIPALLY SOUTHERN ILLINOIS BITUMINOUS.

KIND OF COAL.	SIZE.	B. T. U. per pound.	Theo. Evap. Cap'y	PROXIMATE ANALYSES. Moisture.	Volatile Matter.	Fixed Carbon.	Sulphur.	Ash.
Big Muddy . .	Lump	12,190	12.62	7.18	28.83	55.58	1.39	7.02
" . .	"	12,126	12.53					
Murphysboro .	"	11,766	12.18					
" .	"	11,511	11.92					
Hurricane . .	"	11,455	11.86	6.84	27.57	53.07	1.72	10.80
Mount Olive . .	"	11,481	11.88					
" . .	"	11,352	11.75					
" . .	"	11,281	11.68					
" . .	"	11,278	11.67	10.84	31.63	42.37	5.08	10.08
" . .	"	11,100	11.49	12.04	32.95	41.63	3.28	10.10
" . .	"	11,085	11 47	11.36	33.15	41.08	4.18	10.23
" . .	"	11,024	11.41					
" . .	"	10,980	11.37					
" . .	"	10,965	11.35	9.27	30.87	42.66	5.32	11.88
" . .	Run of Mine	10,930	11.31					
" .	"	11,233	11.63	11.26	30.39	45.91	3.88	8.56
" . .	"	11,130	11.52	11.98	28.01	45.83	4.28	9.90
" . .	"	10,836	11.22					
" . .	"	10,771	11.15	8.58	29.37	41.55	5.32	15.18
" . .	"	10,669	11.05	11.70	28.35	40.85	5.58	13.52
" . .	Nut	11,217	11.61	10.35	30.45	47.29	3.62	8.29
" . .	"	10,512	10.88					
" . .	Slack and Nut	10,578	10.95	12.29	30.26	38.50	4.66	14.29
Glen Carbon . .	Lump	11,481	11.88	10.66	32.74	41.99	4.26	10.35
" . .	"	11,350	11.75	9.66	31.96	41.82	4.16	12.40
" . .	"	11,000	11.39	10.78	33.32	41.55	4.25	10.10
" . .	"	10,836	11.22	10.48	31.64	39.08	4.32	14.48
" . .	"	10,707	11.09					
" . .	"	10,512	10.88					
" . .	"	10,320	10.68	10.52	31.80	39.77	4.21	13.70
" . .	Run of Mine	11,674	12.09	9.78	32.73	44.75	3.47	9.27
" . .	"	11,666	12.08					
" . .	"	11,610	12.02					
" . .	"	11,041	11.43	10.25	31.14	43.03	3.84	11.74
" . .	"	10,686	11.07					
" . .	"	10,707	11.09	10.52	30.96	41.22	4.20	13.10
Collinsville . .	Lump	11,153	11.55					
" . .	"	11,000	11.39	9.46	31.09	40.35	5.82	13.28
" . .	"	10,707	11.08	7.95	31.27	39.20	5.50	16.08
" . .	Nut	10,232	10.59	9.88	30.50	39.20	2.40	18.02
" . .	"	9,721	10.06	9.78	26.62	44.61	1.85	17.14
Bryden Royal .	Lump	10,232	10.59	6.73	32.71	48.56	1.73	10.30
" .	Run of Mine	10,679	11.05	7.46	32.55	43.55	4.66	11.78
" .	"	9,848	10.19	7.06	30.90	48.20	. .	13.84
Heintz Bluff . .	Lump	11,126	11.52	9.26	29.29	43.68	4.42	13.35
" . .	"	11,029	11.42	6.62	31.36	40.60	6.12	15.30
" . .	"	10,815	11.20	9.70	32.88	39.86	4.96	12.60
Superior . . .	Nut	9,848	10.19	6.54	29.74	44.02	3.90	15.80
" . . .	"	9,440	9.77	9.92	29.54	41.76	2.50	16.28
" . . .	"	9,336	9.66	9.80	28.12	42.92	2.66	16.50

Kind of Coal.	Size.	B. T. U. per pound.	Theo. Evap. Cap'y	Proximate Analyses. Moisture.	Volatile Matter.	Fixed Carbon.	Sulphur.	Ash.
Gillespie . .	Lump	9,976	10.33	8.84	29.86	48.45	1.58	11.27
" . . .	"	9,722	10.07	8.60	29.50	50.30	1.36	10.23
Southern Ill. .	"	10.905	11.29	7.41	30.81	43.78	4.13	13.87
" .	"	10,900	11.28					
Belleville . . .	"	11,230	11.62	10.35	29.27	45.65	3.18	11.55
" . . .	"	11,047	11.44					
" . .	"	11,021	11.41					
" . . .	Mixed Lump and Slack	10,320	10.69	8.42	30.24	41.16	3.54	16.64
Rentschler . .	Run of Mine	10,961	11.35	9.87	28.15	45.53	3.98	12 47
St. Clair . . .	Nut	10,578	10.95	10.09	28.15	41.96	4.40	15.40
Wilderman . .	"	10,300	10.66	10.65	26.30	40.66	4.76	17.63
	Miscellaneous.							
Cherokee, I. T. .	Lump and Slack . . .	12,662	13.11	2.—	31.40	50.40	4.50	11.70
" .	" . . .	11,997	12.40	2.12	32.10	49.38	4.20	12.20
" .	Slack	11,675	12.07	3.62	29.51	48.09	4.00	14.78
" .	"	11,353	11.74	4.36	28.35	47.53	4.68	15.08
" .	"	10,671	11.03	4.07	27.67	42.12	5.94	20.20
" .	"	10,662	11.02	3.81	27.51	42.42	5.30	20.96
" . .	"	10,513	10.87	4.28	26.57	42.17	5.62	21.36
Hocking Valley	Run of Mine	11,757	12 17	7.72	28.63	55.72	.60	7.33
Pocahontas . .	Lump	13,029	13.49	1.20	17.28	75.02	.90	5.60
Kansas & Iowa .	Mixed Slack	10,900	11.27	4.83	26.28	45.49	5.48	17.92
	Cokes.							
Connellsville . .		12,850	13.30	.34	.36	87.80	.72	10.78
" . .		12,850	13.30	.38	.27	88.32	.80	10.23
Gas House . .		12,300	12.73	.60	.98	82.74	1.08	14.60

GAS PRODUCERS, AND THE MECHANICAL HANDLING OF FUEL FOR SAME.

By C. L. Saunders, Member of the Civil Engineers' Club of Cleveland.

[Read before the Club, October 27, 1896.*]

Producer gas is rapidly gaining recognition among engineers as one of the foremost means of effecting a saving in the cost and an improvement in the quality of manufactured articles, and is being introduced into many of our large industrial establishments.

During the past year several articles describing new gas producers, and discussing the modifications and improvements of the types now in general use, have appeared in the leading engineering magazines. For these new gas producers the inventors claim several points of superiority and economy, such as an increased yield of a new and more permanent gas containing a greater number of heat units per pound of coal, and capable of being brought to the point of combustion with small loss, as also freedom of the products of combustion from qualities deleterious to the material to be heated in the furnace. The various points of superiority of these producers have been so clearly demonstrated, that the engineer contemplating the installation of a gas producer has only to refer to the various articles pertaining to this subject, or to enter into correspondence with the manufacturers of the different gas producers, in order to have at his command all the data required to enable him to choose—relative to fuel, space, location, and the material manufactured—the best adapted to his purpose. Producer gas is unquestionably the cheapest artificial fuel gas per unit of heat.

The following processes have recently come into extended use:

(1) Gas processes, which are essentially water-gas processes. In these the air and steam are admitted at the top and drawn down through an incandescent bed of fuel by an exhauster.

(2) Fuel-gas processes, which are combined water- and oil-gas methods, effecting the decomposition of hydrocarbons injected in small quantities at a number of points.

(3) Gas Producers, with water-trough for bottoms to permit of the continuous removal of ashes, either by manual labor or by a screw conveyor.

(4) Gas Producers having revolving bottoms to facilitate the removal of the ash and clinkers, which are discharged continuously over the edge of the revolving bottom into a sealed ash pit beneath, without interfering with the making of gas.

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Having decided on the type of producer or the system of gas manufacture, the questions to be considered are: The manner of conducting the gas to the point of consumption; and the methods and facilities for cleaning, involving the character and construction of flues, cleaning doors, man-holes, burn-outs and valves. *All* of these questions are important, and, in the final adoption of the small details, the selection of those best suited to permanency and local conditions means economy and increased efficiency in all future operations. Very often a Superintendent finds that his men have failed him and that he must depend on green and ignorant hands. The liability of this to occur at most inopportune times is a factor never to be lost sight of in designing and erecting this class of machinery. The result of ignorance may be the loss of hundreds of pounds of fuel by burning or waste; while carelessness may be the cause of injury to life and limb and the destruction of property.

A thorough knowledge of the kind and character of coal to be used is a primary necessity. It is not necessary to know what the coal could do under many varying conditions determined by exhaustive laboratory experiments, but it *is* of practical value to know what might be expected from day to day, under the ordinary conditions of coal-handling by cheap and ignorant men unaccustomed to working with gas producers.

Briefly stated, the coal used in any system of gas manufacturing should be a coking bituminous coal of good quality, rich in hydrogen, *i.e.*, in volatile hydrocarbons, and should have a low percentage of ash, which should not clinker nor run together under the influence of the heat. The coal should be used fresh, or carefully stored under cover to prevent atmospheric distillation of the volatile matter. It should be as dry as possible at the time of its manufacture into gas, for two reasons: First, to prevent the loss of the heat, which otherwise would be required to evaporate the moisture; and, second, to prevent the condensation or chemical combination of this moisture in the flues, which would precipitate heavy hydrocarbons (tarry matter). These hydrocarbons contain great heating power, and they would thus be lost at the point of combustion.

The coal should be as nearly as possible uniform in size, as this will make level fires which burn evenly. Dust should not be used, as it would adhere to the sides of the bells and hoppers, and form tar, and would interfere with their operation, or it would be carried by the drafts into the flues and deposited there before complete distillation of gas within the particle had taken place, necessitating, besides the loss, frequent burning or cleaning of flues. Neither should large lumps be allowed, as they would require longer burning than surrounding

material and would make irregular fires, some parts of which would be at a white heat while large-masses of coal would be hardly heated through. Air and steam soon force their way through these weak spots and escape into the gas space above, burning both coal and gas.

When large coal is used the man attending the producer need poke the fire but little to liberate the gas. On the contrary, if clean fine coal is used, a greater amount of care and attention will be required, in order to prevent the coal from coking together and arching over. To liberate the gas, the bed of coals must be constantly poked, and the ashes removed uniformly, to allow the even descent of the bed of fuel.

The character of the hydrocarbons depends principally upon the temperature at which they are produced. Operating at low temperature gives easily condensed tarry matter (liquid and solid hydrocarbons) of considerable heating power ; while with producers operated at high temperature, having the body of the fuel maintained at a bright red heat, the yield of permanent gases and hydrogen is very large, and that of tarry matter correspondingly small.

By the use of a fine clean coal, a large volume of gas will be suddenly generated, giving at the same time a larger quantity of finer and more even ash, which can be handled easily and cheaply. For the production of a given number of heat units a larger quantity of coal will be required, because fine coal is slacked more rapidly by the air, and always contains a greater percentage of ash, than lump of the same quality. With the use of coal having no extremely large lumps, the repairs and delays, incidental to breakdowns, as well as the operating expenses of the coal-handling machinery, are greatly lessened, while the reliability and capacity of the furnace is greatly increased.

The introduction of producer gas into many of our large industrial establishments, and the many new processes of manufacture dependent upon the successful application of gas, have stimulated engineers to devise methods and means to improve all the mechanical construction in connection with the gas manufacture, so as to insure permanency, to reduce the labor of operation and to make the charging and cleaning as nearly automatic as possible, thus reducing the cost of operation and maintenance to the lowest possible point.

There is no way in which this can be more readily accomplished than through a successful plant for handling coal and ash. The most economical and successful plant will be that which best utilizes the force of gravity in its operation. If the conditions would permit the unloading of the coal directly from the cars into the storage bins, from which it could then be carried to the gas producers by the force of gravity, it would be possible to erect an ideal coal-handling plant.

The choice between the several systems of coal-handling machinery

rests with the engineer—each system having its superior and its objectionable points. Preference should be given to that system which has the fewest excessively strong and durable moving parts. Avoid the too common error of building only for immediate needs, and erect the work, even if at a greater first cost, with a view to the *unusual* demands which may be put upon its capacity and strength.

Before deciding on the method of coal handling, it is essential to ascertain the best manner of moving the cars, and to make provision for the storage of both empty and loaded coal cars. Care should be taken to run the grades of the tracks so that when the loaded cars are left by the yard engine, all future moving may be accomplished by simply dropping the cars down a grade, and, when they are empty, leaving them on a side track for removal.

Coal-handling plants may be divided into the following classes:

(1) Plants, beneath whose tracks are coal hoppers, to which the cars are either shoved by an engine or pulled by a cable, and then automatically dumped or unloaded.

(2) Plants having unloading devices by which the coal is lifted out of the cars and carried away.

In the first class we find the following subdivisions:

(1) A single hopper, into which the coal can be unloaded at one point only, provided with a discharge gate emptying into a stationary elevator, which lifts the coal to a conveyor, located over and discharging into storage hoppers above the producers.

(2) Plant as described, except that the overhead conveyor is replaced by a small dump car running on tracks above the storage bins.

(3) A series of hoppers long enough to permit several cars to be dumped or unloaded at the same time, provided with gates opening into a conveyor which carries the coal to a stationary elevator; the remainder of the construction as described.

(4) A series of hoppers or bins long enough to permit several cars to be unloaded at the same time, and provided with a series of gates opening into a traveling elevator, which moves along in front of the bin and elevates the coal into a parallel series of overhead hoppers above the gas producers.

Unloading devices may be divided as follows:

(1) Traveling jib-cranes or cantilever hoists, moving on tracks laid alongside of or elevated above the car tracks. The hoist lifts and operates a patented clam shell or automatic filling and unloading bucket, or a bucket of the ordinary self-dumping character.

(2) Traveling-power shovels which move on tracks laid by the side of, or elevated above, the car tracks and having a drop-bottom bucket-shovel on the end of a movable arm. The bucket is filled by being

forced by the arm down into a coal car. It is then lifted and swung over to one side, and the coal is dumped on the floor, into a hopper, or transferred to a system of conveyors and elevators which carry the coal to its destination.

(3) Unloading-elevators, having a continuous bucket-elevator which is raised and lowered directly over the car and provided with suitable guides in a stationary or movable frame-work, supported by legs on each side of the track. The elevator is lifted, by hoisting-chains attached above, high enough to permit the car being placed beneath. The elevator, while running, is then lowered to the coal car. The front and bottom of the elevator boot being open, the buckets fill with coal as the elevator is lowered, until the buckets just clear the bottom of the car. Then the car is moved along, continuously feeding the running elevator, until the car is unloaded, when the elevator is lifted out of the car.

Any of the classes named, with their several modifications, may be operated by steam or by electricity, and may be combined according to the requirements of the plant, or to the individual preference of the engineer.

Time will not permit me to give a description of the buckets, chains and conveyors which the engineer will have at his disposal for the erection of a coal-handling plant, but he can safely refer to the catalogues of the Hoisting, Conveying, and Link Belt Engineering Companies, and obtain information as to the special work at hand.

Bradley & Poates, Engr's, N.Y.

Editors reprinting articles from this journal are requested to credit both the JOURNAL and the Society before which such articles were read.

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REFRIGERATION.

As Applied to Dwellings, Hotels, Hospitals, Business Houses and Public Institutions.

BY ALFRED SIEBERT, MEMBER, ENGINEERS' CLUB OF ST. LOUIS.

[Read October 7, 1896.*]

UNTIL recently ice has been used in the above establishments exclusively to obtain the necessary refrigeration for preserving food, for making ice water, and for preserving perishable goods.

The ice was mostly applied direct, and, therefore, temperatures lower than 38° to 40° could not be obtained, and furthermore the rooms and food thus refrigerated was always moist, there being no way to abstract the moisture necessarily produced by the condensation of the water vapor in the air.

Later, the ice was mixed with common salt in large tanks, a strong brine resulting, of a temperature of about 10° F. This brine was then circulated by a pump through coils located in the rooms to be refrigerated.

In this manner temperatures of 15° F. and a dry atmosphere in the boxes could be obtained, but the process was expensive, as the salt once used had to be wasted.

The refrigerating machines used at present furnish low temperatures and dry atmosphere at little expense where power can readily be had.

Electricity is, as yet, too expensive, since the cost in small quantities is 6 cents per horse-power hour. Gas engines, with gas at 80

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cents per thousand feet, furnish the indicated horse-power at about 2 cents, while the actual horse-power costs 2½ cents. Gasoline engines furnish power for about 1½ cents per horse-power, but are undesirable on account of danger of fire, and insurance companies therefore object to them.

Where a steam plant is already on hand, with or without an engine, it is easy to produce a horse-power for 1 cent per hour, if the wages of the attendants who are needed for other purposes are not considered, and in such cases it pays to put in a refrigerating machine.

Where, however, no steam plant is available, the only salvation at present is a coal-oil engine, using oil of 150°—220° flash point and furnishing the brake horse-power for 1 cent per hour.

Against such an engine no insurance company will object, and, since it is entirely automatic, it will need no attention for, say, ten hours after once being started.

Even when the load is taken off, the speed changes only at the very moment, so that if the load is taken off after one half minute, an engine whose standard speed is 230 revolutions per minute, will still register only 233.

In very small refrigerating machines we have to allow two or three brake horse-power per ton of ice capacity, and in large machines, according to the temperature of the cooling water, 1¼ to 1½ horse-power per ton of capacity. In ice machines, one ton capacity is the capacity of melting one ton of ice in 24 hours.

When a machine, therefore, is to replace the ice formerly used, it is easy to calculate the expense of power, but it is not so easy to calculate the capacity of a refrigerating machine when certain work is to be done which has not been previously done by ice, and a refrigerating expert should then be consulted.

No general rules can be given because the nature of the insulation, the location, and the size of the rooms to be refrigerated have to be considered, also the number of times doors are opened, and the nature of the material put in the boxes greatly affect the problem.

The next question to consider is that of water.

We need, per ton of refrigeration, two gallons of water, if hydrant water is used, which is about 85° in high summer, and one gallon per ton of refrigeration, if well water from 56° to 60° is used. Now city water, in small quantities, costs about 15 cents per thousand gallons, and, since a one ton machine would use per day 24 x 60 x 2=2880 gallons, the water would cost 43 cents.

But frequently this water can be allowed to run into a tank and can be used for other purposes, such as making steam, washing purposes, flushing closets, etc.

The water can be used for any purpose whatever. It is just as clean as it was before being used by the ice machine, only from 15° to 30° warmer; 30° in the case of well water and 15° in the case of river water.

Having ascertained the first cost of the apparatus, we can now calculate the actual cost of operating it, counting running expenses, interest on capital and deterioration, and can ascertain whether it will be cheaper to refrigerate with ice or with the machine, provided satisfactory service can be obtained with ice.

Frequently, however, the inconvenience of handling the ice, the expense required for storing it, the wet atmosphere and the unsatisfactory temperatures obtainable, outweigh even an excess of expenses.

The refrigerating machines are used for the cooling of rooms, the making of ice, the freezing of water in carafes, the making of ice-cream, and the cooling of air to be injected into living rooms.

We have now to decide which system of refrigeration we shall adopt, the direct or the indirect.

In the first, the direct system, ammonia is circulated through pipes located directly in the rooms to be refrigerated, while, in the second case, the indirect system, the ammonia is circulated in pipes located in a brine tank, cooling the brine, which then in turn cools the rooms, it being circulated through pipes provided in the rooms.

Ice making, carafe freezing, and ice-cream freezing are always done by the indirect system, while the cooling of the air is more economically done by the indirect system, but can be done by the direct system when blown into rooms.

The cooling of rooms direct should be done by the direct system, if possible, or (when it is found necessary to stop the machine over night to save attendance, and when the boxes are small and will be used over night) by a combination system.

To use the indirect system for small boxes when the machine stops over night is no advantage; first, the brine tank must be made considerably larger in order to store sufficient cold brine, which can be circulated through the cooling pipes in the rooms while the machines are stopped.

And, second, in order to circulate the brine, a pump must be kept going, which needs attention and power, so that either a boiler must be kept going or a motor of some kind, which also need attention.

The indirect system for rooms to be refrigerated is really an advantage only when the rooms are very small and the machine runs continually, since it is difficult to regulate the temperature of such boxes by the direct system, the number of running feet of pipe in boxes being so small that the liquid inlet cock cannot be adjusted accurately enough, and either too much liquid or none will enter the pipes.

In the direct system no brine tank, nor brine pump is required, and, while the brine pipes are somewhat cheaper than the ammonia pipes, the price of the brine tank and pumps more than counteracts this difference.

While the life of ammonia pipes is infinite, the life of brine pipes is about three years. The pipes themselves would last longer, but the threaded ends give out, and this necessitates the removal of the pipes just the same.

Further, it is not always easy to provide space for the brine tank and brine pump, and then the eventual leaking of brine tank and the adding of brine are undesirable features.

In other words, there are only very rare occasions where the indirect system should be used.

The combination system, however, is admirably suited to the cooling of small rooms or boxes. It consists of ammonia pipes, which are partly exposed to the air of the box and partly submerged in weak brine.

A galvanized iron trough is suspended from the ceiling and filled with weak brine, and, in the space between ceiling and trough, one, two, or more horizontal rows of pipe are suspended.

In the case of one row, the pipes may be one-half submerged, and in the case of two rows, one row may be submerged and one row exposed to the air.

The trough, must, of course, be so arranged that the air can reach the space above it on one side, pass over the pipes, become cooled and descend on the side opposite to that from which it started.

This is done by making the trough narrower than the boxes, leaving a passage for the air on each side.

If the box is wider than about three feet, it is better to provide two troughs, leaving a double passage between the two troughs and a single passage on each side.

The warm air will then ascend through the central double passage, and divide, one half passing over each trough and descending through one of the single side passages.

Of course, sufficient spaces must be left for the free passage of the air.

The advantages derived by the use of the combination system (Richmond Patent), are the following:

First. Easy regulation of the liquid. When it is the intention to run the machine only 12 hours per day, sufficient pipes must be provided to do the cooling required for 24 hours in 12 hours, and, therefore, twice the amount of pipes must be erected, consequently the range for the liquid to act upon is increased.

But, since a part of the pipes (the part further away from the inlet) is submerged, the contact with the medium to be cooled is better, and considerable more heat can be transmitted by each foot of pipe, hence there is little chance for the liquid to leave the coils as such, and accumulation of refrigeration by the cooling, and much more by the freezing of the weak brine, is obtained. The brine is made weak so that it will freeze at about 28°. It can then easily keep a box at 34° as long as it is melting, and, even melted, the brine itself, as a liquid, can absorb heat until its temperature reaches 34°.

It will be seen that in this manner a storage of refrigeration is obtained, which is very convenient, takes up very little room, and works automatically, storing whenever there is surplus refrigeration.

The making of ice is rather a cumbersome affair, and does not pay in small quantities.

Ice made of raw water has a snowy appearance, and the water must be carefully filtered in order to avoid coloring.

A distilling apparatus takes up much room and is expensive. Besides, the vapor rising from the steam condensers and reboiling tanks must be carried off, and this is seldom convenient in a dwelling house.

For hotels, however, where it is easy to obtain distilled water, there being quite a large quantity of surplus exhaust steam on hand, it pays to make ice.

In dwelling houses, freezing the water in carafes can easily be accomplished, no distilling appliance being necessary, since the appearance of the ice formed in the carafes is not objectionable; and, if a little opaque ice is wanted, small ice cans can be placed in the pockets provided for the carafes and the water therein frozen.

A small place can be reserved for the ice-cream freezer, which need be of no special construction, and the motor which is used to circulate the brine in the carafe tank can be connected to the stirring apparatus of the ice-cream freezer.

The cooling of the air for living rooms, fever rooms, hospitals, libraries, etc., is another useful and important application.

There the condition of the atmosphere in regard to moisture plays an important part.

It is well known that air and water vapor exist together in our atmosphere, the percentage of water vapor varying from 35 to 100. According to Dalton's law, two gases or vapors like the above exist together, just as if the other was not there, the temperature of the mixture determining the pressures and in consequence the weights of the water vapor, provided the percentage of the moisture has been obtained by a hygrometer of some kind.

In one cubic foot of air there exists therefore one cubic foot of

water vapor. Assuming the temperature of the air to be 85° we find the pressure 0.592 pounds and the weight of one part of it 0.00182 pounds. If now the percentage of moisture is 60, we know that this cubic foot of mixture contains $0.00182 \times \frac{60}{100} = 0.00109$ pounds of water vapor.

From the above and by the use of a steam table it will be readily seen that if we cool air, we increase rapidly the percentage of moisture, finally condensing part of it, which is, however, a very objectionable occurrence in a living room or a fever room.

If, therefore, we do not want to abstract the moisture by heating the air after having cooled it below the desired temperature and lost water vapor by condensation, we must be satisfied with a reduction of about 10° F. in the temperature of the air.

For instance, if we take the previous example and fix 75 as a limit for the percentage of moisture permissible, we find that taking 0.00109 as being 75 per cent. of the weight of one cubic foot, that one cubic foot would weigh 0.001336 pounds, which would correspond to a temperature of the cooled air of 75° (see steam tables). If we should cool the air any further the percentage of moisture would exceed 75, and this is not permissible.

Taking, now, the case that we are required to furnish air at 60° F. containing only 50 per cent. moisture, the air to be cooled having a temperature of 95° and a maximum percentage of moisture of 75, the question is, how low must we cool the air in order to condense sufficient water vapor to obtain the proper amount of moisture when the air is heated again to the temperature desired.

We have air at 95° and 75 per cent. moisture, consequently each cubic foot of air contains (see table) $0.00245 \times \frac{75}{100} = 0.00184$ pounds.

We have air at 60° and 50 per cent. moisture. Each cubic foot of air contains then (see tables) $0.00082 \times \frac{50}{100} = 0.00041$ pounds. of moisture, or we have condensed $0.00184 - 0.00041 = 0.00143$ pounds

Since we want to retain only 0.00041 pounds of water vapor, we must cool the air to such a temperature that, when saturated, it contains 0.00041 pounds. Consulting the table, we find that this takes place at 40° F.

To recapitulate, we have first to cool the air of 95° to 40°, and then to heat it again to 60°, in order to obtain the desired conditions.

The work of refrigeration required for this is as follows:

Neglecting the fact, that owing to the pressure of the water vapor (about one-half pound), the pressure of the air itself is not quite atmospheric pressure, the sum of the pressures of the water vapor and the dry air being equal to the pressure of the atmospheric air.

We can divide the work into four parts:

1. To cool the dry air.
2. To cool the water vapor.
3. To condense the water vapor.
4. To freeze the water vapor when the direct system of cooling is used.

1. AIR COOLING.

Weight of cubic foot of dry air at 95° = 0.0728
Specific heat of dry air at 95° = 0.2377
One cubic foot requires 0.0728 × (95° — 40°) × 0.2377 = 0.941

2. WATER VAPOR COOLING.

Specific heat of water vapor about 0.5
One cubic foot requires 0.00041 × (95°—40°) × 0.5 = 0.012 th. u.

3. WATER VAPOR CONDENSATION.

Average latent heat = 1000 th. u.
One cubic foot requires 0.00143 × 1000 = 1.43 th. u.

4. WATER FREEZING.

Latent heat of congealing 1.42 th. u. 0.00143 × 143 = 0.203 th. u.

2.586 th. u.

It will be seen how small is the amount of work required to cool the air direct, and what an important role the water vapor plays. It will be further seen that the indirect cooling system is preferable for air cooling, and that only water should be used as the circulating medium, because then there can never be obtained a temperature of air below 32°, and no freezing of the condensed vapor can take place.

Besides, it is much more economical to have a medium at higher temperature, say, about 34°, because then the refrigerating machine can be worked with about 40 pounds suction pressure, while, if the brine of 20° were used the machine would have to work with about 25 pounds sunction pressure, which would mean that in the first place with an expenditure of about 2 per cent. more fuel, 37 per cent. more refrigeration can be obtained. Twenty-five pounds pressure is the usual suction for machines working on the direct and indirect cooling system, also when freezing water in carafes.

The air cooling is generally combined with the heating of the building, either by steam or water.

The radiators are located in air ducts and air is blown over them. In winter, steam or hot water is circulated through them, and in summer cold water, if necessary. Of course an extra radiator for reheating the air must be placed behind the radiator cooling the air; but, as already explained, this will be necessary only when lower temperatures are wanted.

For ordinary living rooms, it is considered that furnishing air cooled 10° four times every hour will have a very agreeable effect, and will purify the air in the room.

RECENT PRACTICE IN RAILROAD SIGNALLING.

By George W. Blodgett, Member of the Boston Society of Civil Engineers.

[Read before the Society, September 16, 1896.*]

The last paper on railroad signalling read before this Society seems to have been that presented, in 1886, by Mr. G. R. Hardy, who presented the subject in a general way, with special reference to the then recently installed system of interlocking for the junctions and terminals of the four tracks of the Boston & Albany Railroad, from Boston to Riverside, while the writer treated of the automatic signalling of the remainder of that portion of the road.

It is not the intention of the author to review any considerable part of the contents of these papers, but for the benefit of those who have more recently become members of the Society it may be desirable to refer particularly to some of them.

It is the purpose of the writer, however, to confine himself, in the main, to the consideration of the progress that has been made in the last ten or twelve years in railroad signalling and of the standard practice of to-day so far as it is well established. He will first take up interlocking at terminals and junctions, and afterwards refer to block signalling, both manual and automatic.

So far as the writer knows, the Saxby-Farmer interlocking machine was the basis and the general type of all the machines for mechanical interlocking which are in use in this country to-day. For the benefit of those who are not at all familiar with the subject, he will say that this machine consists of a series of pivoted levers standing in a nearly vertical position side by side in a frame, and having a motion forward and back in a vertical plane to the extent of about 45° to 60°. All the switches in the vicinity are connected by rods to some of these levers, all the signals (usually by wires) to certain others, while the remainder of the levers control facing-point locks and detector bars, the object of which is to prevent the movement of a switch while a train is passing over it.

There is also a series of sliding pieces called locking-bars,—one for each lever—and other parts, either sliding or rotating, according to the style of machine. To the locking-bars are attached lugs or tappets, which are so arranged as to engage with the sliding or rotating pieces belonging to other locking bars and prevent their motion, except at particular times. Fig. 1 is a front view of such a machine, Fig. 2 an end view, and Fig. 3 a view of some of the details of the locking gear.

* Manuscript received November, 1896.—*Secretary, Ass'n of Eng. Socs.*

The levers are arranged in certain groups or combinations. These belong to consecutive movements which are required for the preparation of particular routes. Out of all the levers in the machine (of which there may be five, ten or a hundred) only a few, it may be half a dozen, can be moved; while all the rest are locked fast in their normal position. Those that are free control the first step necessary in the preparation of the several routes, which may or may not conflict with each other. All of those which do not conflict may be left open at the same time without danger of collision. Of the others, only alternative routes can be used; the selection of one, and the movement of the first lever of the series belonging to it, locks fast all routes which would be inconsistent with it.

So long as trains obey the signals it is practically impossible, no matter what the signalman may do, for him to so arrange the switches and signals that each of two trains can be given at the same time a signal to pass over the same piece of track.

Which levers shall be used, and which shall be locked, and which unlocked by any proposed arrangement of switches and signals, is determined when the machine is set up by a study of all the train movements to be provided for and of their relation to each other. It follows, therefore, that each machine must be designed for the place where it is to be used; and the work it is to do must be known and provided for beforehand.

Where trains are run by telegraph, all the traffic on a division is under the control of the train despatcher of that division, and so, at a junction or yard protected by interlocking, it is a vital part of the system that all the movements must be under the absolute control of the signalman.

Still further, in any of the well-known systems of block signalling, a clear signal means only that the train *has permission* to proceed; whereas, in an interlocking system, the clearing of the signal indicates that the movement it controls *must* be made, and at once, in order to clear the track for others.

The semaphore is still the form of signal ordinarily used, and, except in a few cases, every train movement has had a signal belonging to it which could not be used for any other.

Formerly, only the two switches of a cross-over were attached to the same lever, each thing to be done requiring a separate lever. The changing of a single switch then necessitated three movements (or *five* if the signal be included), and, as the limit of a signalman's capacity for rapid motion is soon reached, in a machine where there are many levers, a second, third, or even a fourth man soon becomes necessary, each working at a different part of the machine, in order that several train move-

ments may take place at once when they can afely do so. This enormously increases the expense, and decreases the safety of operation to a certain degree; and furthermore the locking becomes more and more complicated.

During the last ten years it has become more common, in large machines, to diminish the number of levers required by attaching, where practicable, other movements in the series; such as connecting to a switch lever the lock and detector-bar which were formerly attached to a separate one. Such a switch lever has three distinct functions in different parts of its stroke; first, the switch is unlocked, say by the first third of the movement; the second part of the motion changes the switch, while the remainder of the stroke locks the switch in the new position.

In some machines a still further function was added to the lever, viz., the clearing of the signal; but, as the practice seems to have been entirely abandoned (in mechanical plants at least), I judge it was found impracticable or unreliable.

A valuable addition to the advantages gained by this grouping of functions, is that the locking is greatly simplified, with a corresponding gain in safety and economy of maintenance.

It was formerly the practice not to require the locking of switches for back-up or switching movements, so that flying switching was possible with such an arrangement of interlocking, and there are still in use many plants in which this can be done; but here is a weak point, and, as the pressure of a constantly growing traffic increases, there is more and more danger that a signalman, in a hurry to provide for the next movement, may, by first putting the signal to danger, throw a switch before a train has quite cleared it, and thereby split the train, or that a wheel with a sharp flange may strike the point of the switch and crowd it open. The later and better practice is to lock the switch in all cases where a train is to run against the point.

Another great saving in the number of levers required in a mechanical machine is rendered possible by the use of "selectors," whereby one lever is made to move, one at a time, several signals belonging to converging routes, the particular one to be moved being determined by the arrangement of switches and tracks composing the route to which the signal applies.

This is accomplished by a sliding link or loop in a frame, over which are suspended pivoted bars, each terminating in a hook, so arranged as to engage with the link, if allowed to come into contact with it. The opposite end of each of these bars is connected to some signal in the series, by a chain or wire; all the hooks but one are raised out of contact with the link by cams on rotating pieces con-

nected with the rods operating the switches. One lever moves back and forth in the frame the sliding link, and with it the signal at that moment connected to the link by its hooked rod.

As each route is prepared, the cam attached to the switch rod rotates and allows the hook to engage with the link so that the particular signal desired, *and no other*, will be operated.

A single signal is also made to serve several diverging routes by means of an indicator, consisting of a series of numbers or letters, and placed on the signal post.

Fig. 4 shows such a signal in the all-clear position when route No. 2 is open. The signal would show the same for any other route, except that the figure "2" would be replaced by that corresponding to the route which was then open.

Fig. 5 shows the arrangement of the arms which would have been necessary for the tracks shown in the figure when signalled in the old way. A comparison with Fig. 4 will make at once apparent the superiority of the new method over the old.

When the signal is in the danger position the numbers are all covered by a screen, but when a route is prepared and the signal drawn clear, the number designating that particular route comes into view below the screen. This is accomplished by attaching to the switch lever a device which operates that particular number or letter, but no other.

These two improvements, viz.: the attachment of several pieces of apparatus to the same lever and the use of selectors, have effected, in the later installations of mechanical interlocking, a saving, in the number of levers required, of nearly or quite one third, as compared with the standard practice of a few years ago. In some recent cases coming under the writer's observation, the actual saving was as follows, viz.:—

(1) The re-building of a machine already some years in use reduced the number of levers from 60 to 36, or 40 per cent., although some additional switches and signals were connected to the machine.

(2) A machine where 52 levers are now used would have required, under the old system, 70, or nearly one-third more.

(3) In another case, for the work now done by 17 levers, 32 levers, or 46 per cent. more, would have been needed.

In a fourth case, a machine which now has 24 levers would formerly have had 36, or 50 per cent. more.

These cases, taken at random from a list which might easily be made a long one, serve to show to what extent interlocking machines have been simplified by the adoption of the improvement mentioned and by certain minor details of construction. The saving in first cost is considerable; the decrease in cost of maintenance is on the right side of the ledger; the saving of time is important, but the chief gains are

in the greater simplicity and safety of the locking, which are made possible by these changes of construction.

In interlocking switches and signals in a yard an important advantage is gained by the use of *slip switches* and *movable point frogs*, whereby a single track running diagonally across any number of others, is made to take the place of the large number of cross-overs which would have been necessary under the old plan. [As shown in Fig. 6, where a double slip switch is inserted at every crossing of a straight track by the diagonal ave.]. A cross-over occupies from one hundred and fifty to two hundred and fifty feet in length, but slip switches can be put in a much smaller space. The *exact length* will, of course, depend on the distance between the tracks and on the angle chosen for the diagonal track or "ladder," as it is technically called, which last also depends, in many cases, on the room available.

In many yards all the available space is already utilized, and more cannot be had at any price. In such cases, a growing traffic can be provided for only by the use of every means of saving time and labor. Not the least of the benefits gained is the extreme facility afforded for the transfer of cars or engines from one track to another in making up outgoing trains or in removing empty ones.

The labor involved in the movement of switches and signals in a mechanical machine is so great that in a busy yard, where several operators are required, each works at a different part of the machine, and thus controls a different section of the yard, but in many movements two or more operators necessarily work in conjunction. Generally, several different train movements may be going on at the same time, being under the direction of different operators, the machine itself rendering impossible any unsafe combination of two movements.

The only way yet devised of turning this labor into a pastime and making the work of a signalman so light that a child has all the strength necessary to perform it, is the electro-pneumatic system of interlocking lately introduced at a number of important junctions and terminal points.

The principles of a good signalling system are preserved by providing the machine with a system of locking as complete and thorough in all its parts as in a full-sized mechanical machine, but on a diminished scale, say not more than one-fifth or one sixth as large. The first machines set up had no locking. Any of the levers could be moved at any time, and in any order; but, unless moved in the order determined upon in the arrangement of the parts of the machine, no effect would be produced. Hence, an operator had to learn by practice or by arbitrary memorizing which levers he must or could use.

In the electro-pneumatic system the levers of the Saxby-Farmer

machine are replaced by small cranks having a rotary motion through an angle of, say, 60°. These are, in reality, only electric switches. The signals, which are full-sized semaphores of the ordinary pattern, are connected to a piston moving in a cylinder and actuated by air under pressure, which is supplied by a pump to a reservoir, and thence distributed about the premises by pipes to the cylinders of all the switches and signals in the vicinity. The valves which admit air to the cylinders are operated by electric currents, which are themselves controlled and directed by the changes of circuit produced by the manipulation of the cranks at the machine.

If the operator turns a crank, the first part of the motion simply completes a circuit through an electro-magnet at the switch. This opens the valve and allows compressed air to enter the cylinder at one end and push the piston to the opposite end. This changes the switch, and, at the end of the stroke, a very ingenious device locks it in the new position. Until this has been done, the further motion of the crank is prevented by a stop, which is removed by a return current from the switch *after its movement is finished*, and not before. The completion of the movement of the crank may now be made. This sends a current of electricity to the proper signal, changing it from the danger to the clear position.

In front of the signalman is a small-scale model of the tracks and switches in that part of the yard which the machine operates, and on this are repeated all the changes that take place in the tracks in the yard itself; so that the signalman has constantly before him a diagram of the actual position of the switches and tracks. Should a switch fail to operate, or even fail to be properly locked, no return current would be sent and no signal could be given; also, no change would take place in the tracks of the model.

Fig. 7 shows a front view of an electro-pneumatic interlocking machine, and Fig. 8 a rear view with the casing removed.

The latest and best example of this system is at the Union Terminal Station, in this city, and it needs only a visit to "Tower A," on the Charlestown side of the drawbridge, to enable one to see how marvellous is the progress that has been made.

This work is especially interesting, because the larger part of the apparatus is located on one side of the channel, while the machine which operates it is on the other side; yet it works as well as if close at hand. It is difficult to see how any other than electrical means could have been successfully used for the multitude of controlling or operating circuits which lead out from this tower and pass under the channel, but which must not be permitted to obstruct the water-way.

Derailing switches have not come into general use, and, so far as

the writer's observation has gone, they develop, when used in the ordinary manner, at a junction or crossing, a danger nearly as great as that against which they are supposed to guard. In several cases that have come to his knowledge the derailed train has blocked the tracks when it might have gone over in safety except for the derailing switch.

The much discussed and still unsettled question of the proper lighting of semaphores at night has received its due share of attention, but there is at present no more sign of agreement on this subject than formerly. It is still common to use a red light for danger and a white one for safety in spite of the fact that a white signal light can easily be confounded with another light seen in the same general direction, especially in crowded towns; and that more than one case is known where a red glass broken out of a semaphore has shown a white light when the signal was at danger, and where a wreck was the result. Were green adopted as the color for safety signals, such a derangement as the breaking of a glass could never lure a train into danger, because it could never produce the safety signal. The Railway Signalling Club, with headquarters at Chicago, has given much consideration to this subject in the last two years, and, at the end of a long report, it recommends that red be the standard color for danger in all stop signals and white the standard for safety, because its members see no prospect of bringing about uniformity except by abandoning green for safety signals, although they practically admit its superiority over white; they intimate, however, the breaking of a glass is so improbable, that it is not worth while to provide against it.

Electric locking is used to some extent, but has not grown into the general favor that at one time seemed likely to greet it. Some of the first installations were so contrived that when a signal was drawn clear it was locked in that position. So far as the author knows, he was the first to call attention to the dangerous character of this practice, and to maintain that nothing should ever be attached to a signal to lock it in the all-clear position, and that it should always be in the power of a signalman to put the signal instantly to danger and stop a train, should it be necessary in order to avert a collision, even though the train had begun to occupy the route over which the signal gave the right of way.

The signalman, however, should not be able to change any of the switches, or to interfere in any way with the route until the train has passed, and here comes in what the writer conceives to be the legitimate function of electric locking, viz.: to secure to a train a perfectly safe passage when once it has begun to occupy a route, but to leave it still under the control of the signalman until the critical point be

passed. Hence every signal should be so arranged that it can be put instantly to danger, while the switches remain locked fast, should the train have accepted its right of way, until they are passed in safety.

In manual block signalling the author knows of no changes in practice worth mentioning, but the "Syke's Lock and Block System" has made commendable progress. In this system each man's signal lever is provided with an electric lock, which is applied automatically, but which can be released only by the signalman in advance; and the signalman, when he has once unlocked the signal in his rear, in order that it may be set clear, cannot again do so until the train already in the block has gone out. For this he must set his own signal clear, and this requires his lever to be unlocked by the man next in advance. When this signal has been cleared, and restored to danger behind the train, he may again unlock the lever of the man in his rear, but it is practically impossible for him to do so before.

There is required, therefore, the combined action of two men to get a train past a signal, and that of three men to get it through a section. Used in its integrity, there is almost no possibility of a collision.

The general arrangement of this system is shown in Fig. 9, which is a diagrammatic view of three blocks of a double-track railroad equipped with signals. Fig. 10 shows a front view, and Fig. 11 a side view of the machines in common use.

This system was applied to fifty miles of the New York, Lake Erie & Western Railway several years ago, but was abandoned, after a year of trial, because the blocks had been made so long that a train could not run from one signal to another before a following train would be waiting to enter the block. It has been put on the New York, New Haven & Hartford Railroad from New York to New Haven, and on four tracks of the New York Central from Albany to Buffalo, where it works well. It is far safer than a simple block system, but costs more for its maintenance. Indeed, the expense is the chief objection that can be brought against its adoption. Also, the fullest degree of safety would require the complete equipment of the track with rail circuits, thus making it still more costly, but with a corresponding gain in safety.

It is in the use of automatic signals that the chief advance has been made in the last ten or twelve years.

In two important particulars solid ground has been gained, and there is little fear of the reversal of present practice in regard to one of these particulars, or of the failure to ultimately adopt the other. Firstly, it is now well established that a continuous rail-circuit is a vital part of a first class automatic signal system; and, secondly, in an ideal system the signals should stand normally at danger and be cleared by

the train in advance of itself (provided the block is unoccupied, the track continuous and the switches all set for the main line), and set to danger again as soon as the train reaches the block.

Any derangement which puts such a signal out of service leaves it showing danger, and not all clear, as was formerly sometimes the case, thus giving a possibly false indication of security when, in reality, the apparatus is inactive.

The author's continued experience in another direction has served only to confirm the judgment he formed many years since, viz., that it is exceedingly important that the engineman of an approaching train should actually see the signal move from the all-clear position to that of danger as he enters the block, and hence that the point from which the operation of the signal takes place should be somewhat in advance of it.

The author is well aware that this view is not accepted by some railroad managers of the highest authority and of wide reputation in other directions, but without much familiarity with automatic signals, and while he respects their opinions he has seen no good reason to change his own, fortified, as it is, by a long practical experience and corroborated by what he considers a very significant fact,—viz., that while he knows of changes in practice from so placing signals that they should *not* be seen to operate, to locations where the change would be visible, he has not heard of a single case where a change has been made in the contrary direction.

The fact that a signal is to be worked by a train and not by hand renders it wise that it should be under as strict surveillance as possible, and hence its operation for every train that passes should be observed. An engineman can do this with no appreciable addition to his labor (since he must observe the signal at all events), and he is vitally concerned. He is usually a man of greater capacity than most of those on the train with him, and the author believes that where this practice is the rule, the enginemen themselves would be the most unwilling to change it. Should all work well, there is nothing for the engineman to do; but if any signal fails to operate normally, he may, in a single moment, note the fact, and at the next convenient point have the information transmitted to the proper parties. Much valuable time is thus saved, and more efficient service obtained without any perceptible increase of labor or of expense.

In conclusion, the author thinks that it may be confidently asserted that in no single direction can a limited sum of money, available for improvement in the service of a railroad, be more wisely spent than in the erection of automatic signals. One of the best can be erected at an average cost of perhaps $500, and maintained at not

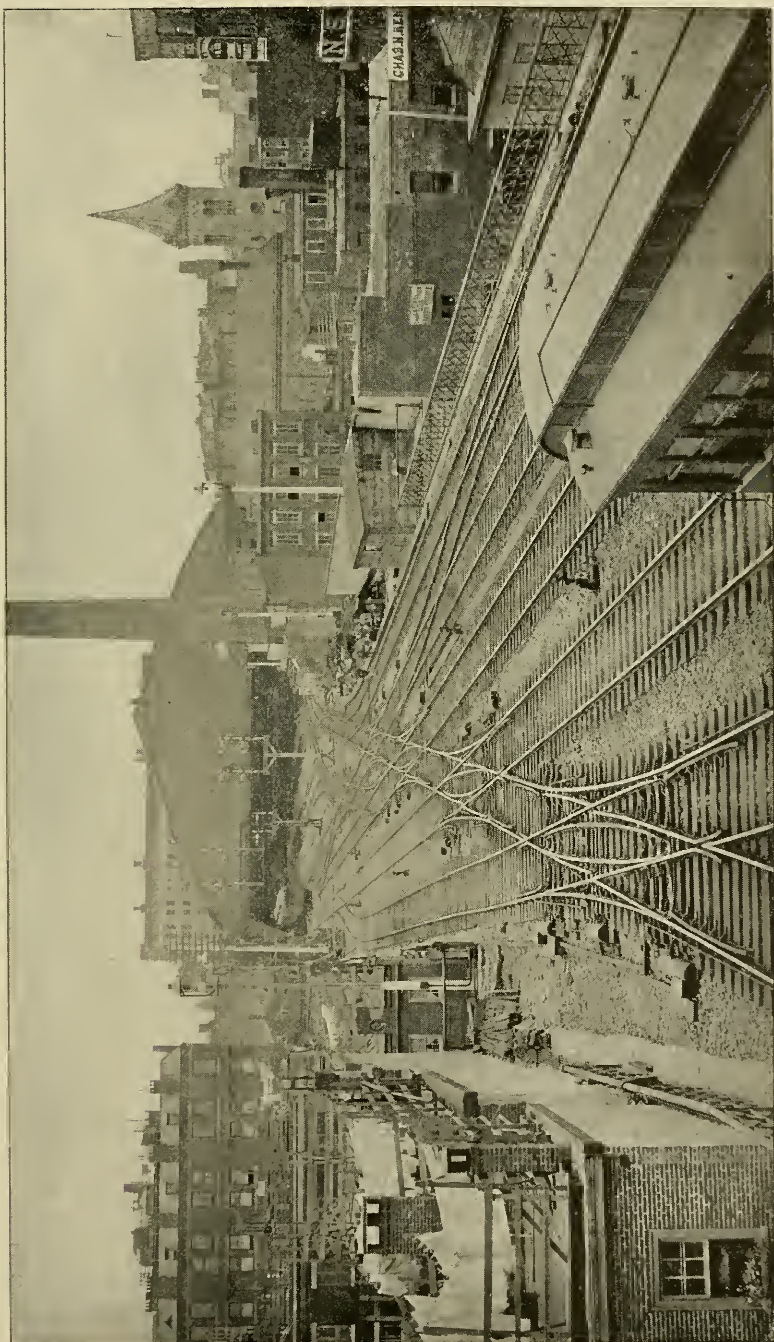


Fig. 6.

exceeding $100 per annum. The average cost in his own experience has been considerably less than this.

A very slight derailment or collision avoided would pay the yearly expense, and a single freight car saved would pay for the installation.

DISCUSSION.

Prof. C. Frank Allen.—There is one point touched upon by the author in regard to which I disagree with him, viz.: the use of the derailing switch. It must be admitted that where there is no derailing switch, a train which runs by a signal may often go on its way without causing an accident, while if there is a derailing switch, the train will go into the ditch, but I do not think this a proof that a derailing switch should not be used. Such a view fails to take sufficiently into account the element of human nature. Many an engineer would sometimes run by a signal if he thought there was little probability that any harm would result from it. It is human nature for a man to do that. But an engineer would, as a rule, be very careful not to run by a signal if he knew that he was sure to run through the derailing switch and into the ditch. When a train goes into the ditch, the railroad officials know about it; whereas, if a train has simply run by a signal, they probably will never hear of it. The very fact, mentioned by the writer, that a train may run by a signal without meeting with an accident, is, if the engineer understands it, an argument in favor of using a derailing switch. It is true, of course, that when running at great speed, a serious accident will probably result if the train does run through a derailing switch, but, the greater the speed, and the more serious the probable result, the less is the liability of the engineer to make a mistake. If the failure to observe the signal is sure to result in a serious accident to the engineer and to others, and if he knows this, it is probable that he will be more likely to run by a signal than to deliberately run into a train squarely in front of him. That railroad discipline can be counted upon to secure the proper observance of signals, is only partially true. A very serious accident recently occurred on the Pennsylvania Railroad, where this discipline must be better than on most other railroads.

Mr. E. K. Turner.—I can hardly agree with Mr. Blodgett in his condemnation of derail switches at crossings. From practical experience with several of these devices, extending over several years, I have found that after the derail switches had been used long enough for the men to become accustomed to them (that is, to learn that they were in the track), very few derailments occurred. The knowledge of the fact that, if a signal at danger is passed, the engine will certainly be de-

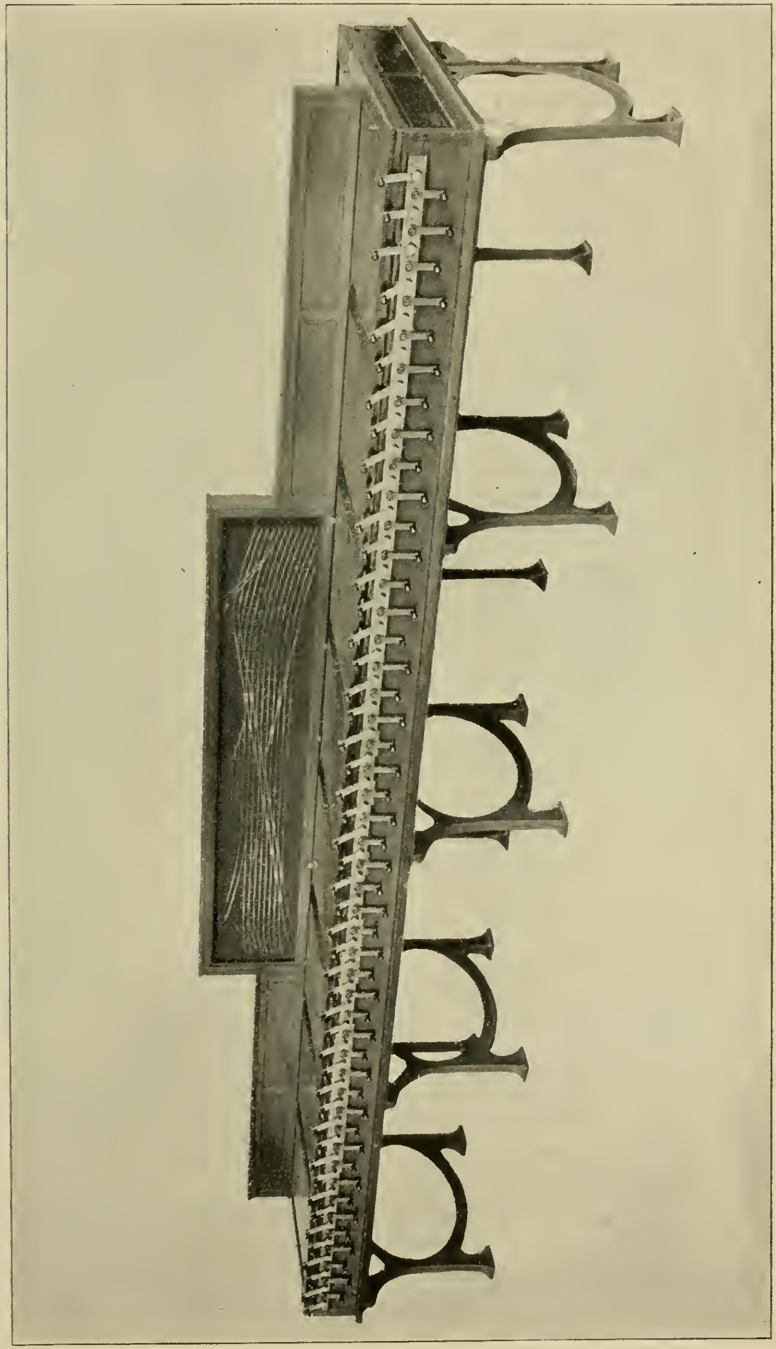


Fig. 7.

railed, makes the engineman more careful in observing the signals. Thus the moral effect is great.

If an engine is derailed it makes its own record and the officials need not ask for or depend upon the report of any employee concerning the violation of rules by another employee.

Of course, with the use of derail switches it is necessary to have strict discipline, every man must have the certainty that a derailment, caused by his failure to observe signals, or to obey orders relating to them, will be followed at once by punishment. If this is relaxed in the slightest degree, the moral effect is lost.

Mr. George F. Sampson.—In discussing this subject of derails it seems to me proper to mention the use of torpedoes connected with an interlocking system, and placed on the rails automatically so that they are exploded by the locomotive and give warning if any train improperly passes a signal set at danger. I am informed that they are used extensively in France, where derailing devices are seldom used and that they are looked upon with considerable favor by some of the best authorities on this subject in our own country. It would seem to me a fruitful field for investigation.

To be sure, an interlocking system is a system of signals designed to prevent accidents, and it is, on that account, best to have the signals observed rigidly according to rule, without being too much complicated with additional devices to tax the mind of the locomotive engineer; but I believe that we should have in mind, as the first object to be obtained—the prevention of all accidents of every kind, including derailments, even if it should prove impossible to detect an offender, a condition of affairs, however, which seems hardly probable.

I am told that, in France, torpedoes are automatically connected with each signal. While I do not see the need of such extensive use, it seems to me that the lever and its fixtures, in an interlocking machine, ordinarily used to throw a derail, could be put to better use in operating a device for placing torpedoes on the rail at such distance from the point of danger as to give warning in addition to that given by the signal arms.

It strikes me that there are but few, if any, men employed as engineers who would fail to bring their wandering senses back to duty with such warning given, while cases are quite numerous where men, having served without accident for a working lifetime, have lost their lives through failure to observe a signal.

Mr. Turner.—Regarding the use of torpedoes in place of derail switches, such use is better than nothing, but does not give that certainty of protection from collisions which the derails give. With the use of torpedoes, it is difficult to locate the violation of rules.

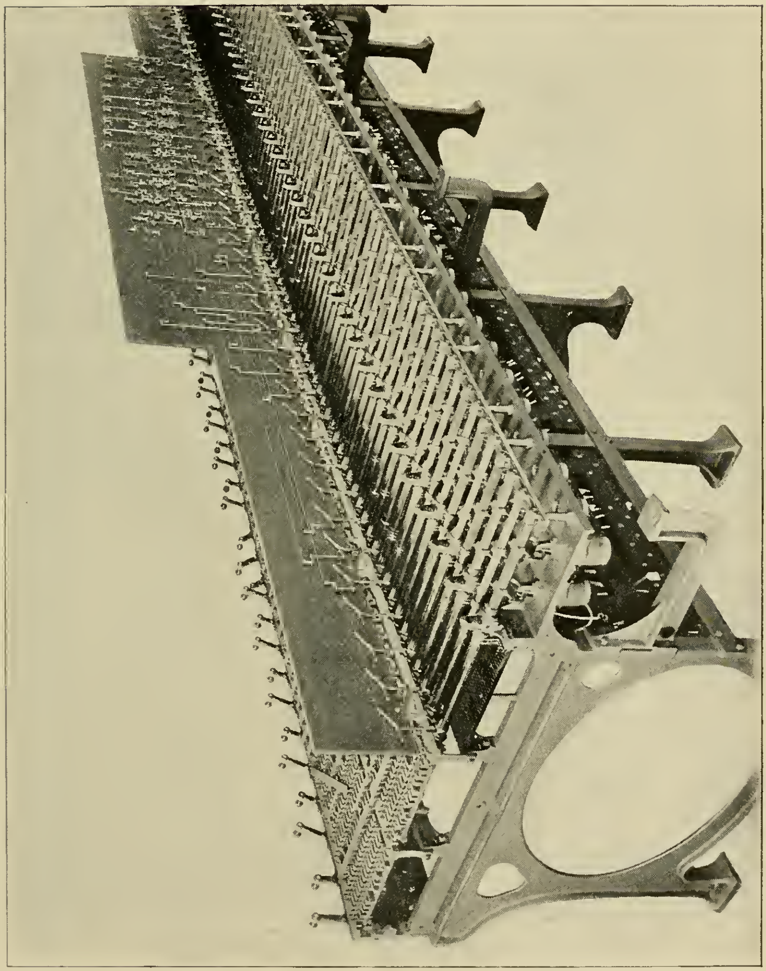


Fig. 8.

In many cases the bad results of derail, noted by Mr. Blodgett could be prevented by the use of sand-covered rails, or of a spare track on to which the engine might be switched instead of on to the ground. But I believe that, in most cases of crossings which are to be passed over by the trains without previously coming to a stop, the safe course is to use the derail switch, and that some of the bad crossing collisions which have occurred would have been prevented by such use.

Unfortunately for the derail switch, the prevention of accident is rarely credited to the device, while the annoyances caused by throwing an engine into the ditch is sure to be charged to it.

FIG. 4.

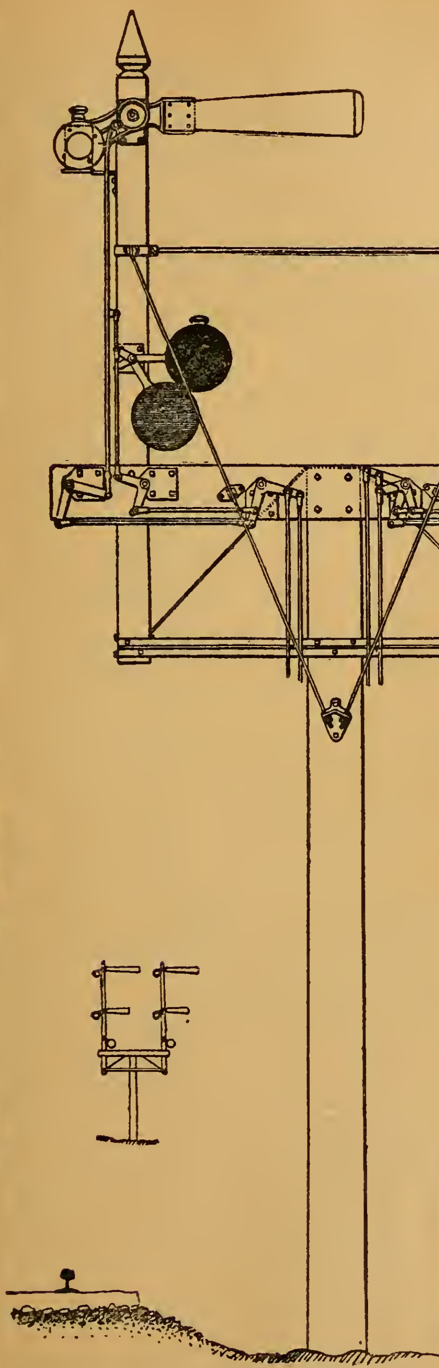


FIG. 5.

FIG. 9.

DIAGRAM OF SYKE'S SYSTEM
APPLIED TO DOUBLE TRACK

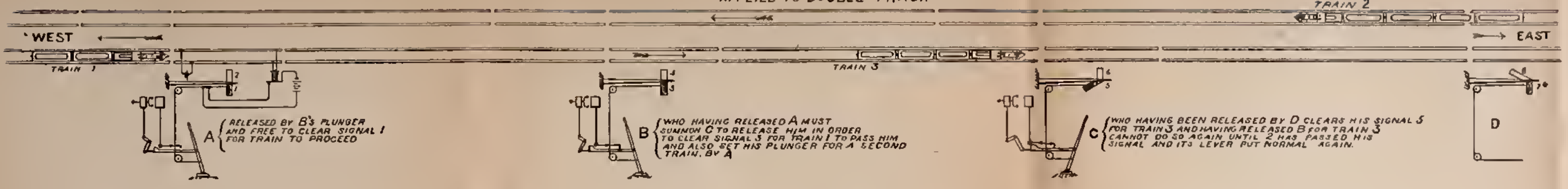


FIG. 1.

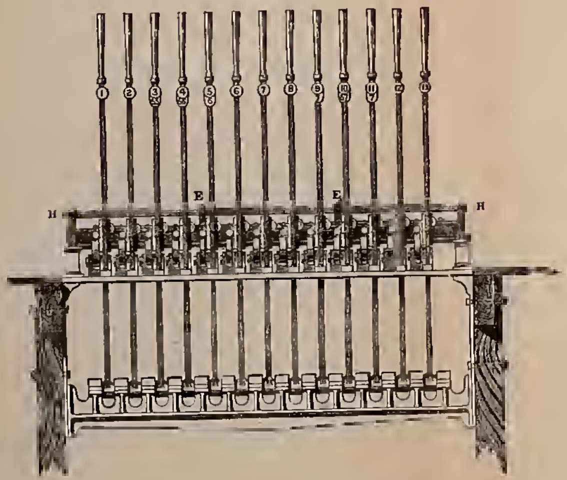


FIG. 2.

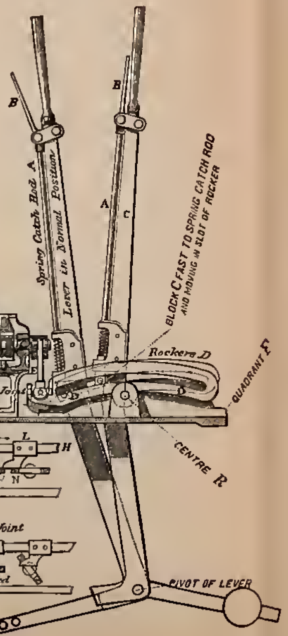


FIG. 3.

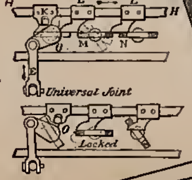


FIG. 10.

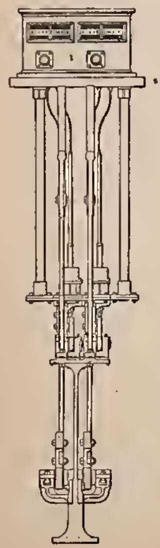


FIG. 11.

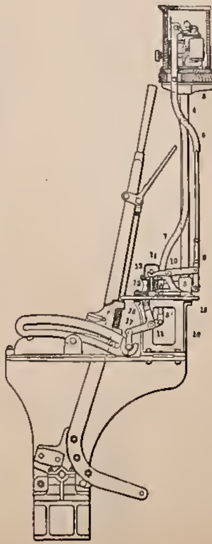


FIG. 4.

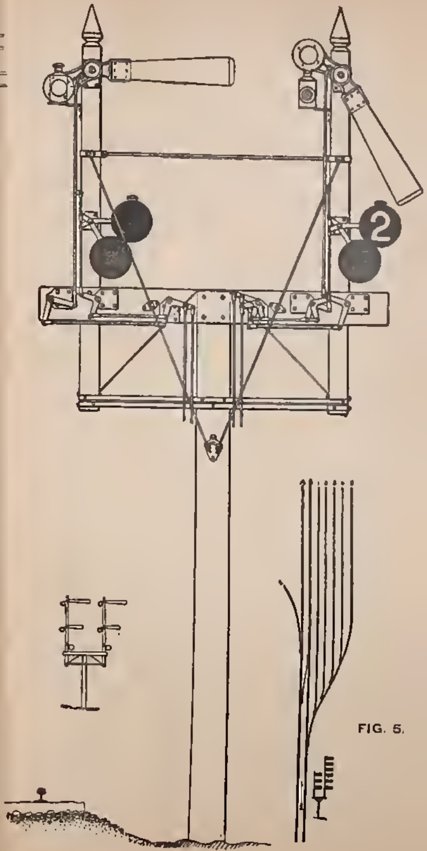


FIG. 5.

THE GALVESTON HARBOR WORKS.

By W. J. Sherman, Member, Engineers' Club of St. Louis.

[Read before the Club, September 16, 1896.*]

The quaint little island city of Galveston, at the northeast end of the long, low and narrow ridge of sand bearing the same name, was once the base of operations of the bold pirate Lafitte. History records no appeals for harbor improvements during that early period; but there seems to have been sufficient water on the bar for the movement of his piratical fleet.

Texas was a State of Mexico, and commerce made few demands upon the rivers and harbors along the barren coast. Later, the people of Texas organized themselves into an independent republic, and commerce began to thrive. To-day they comprise a State of the American Union: this State, an Empire in itself, as great as the combined area of Ohio, Indiana, Illinois, Michigan, Kentucky and Tennessee, and possessed of resources unlimited, which, under the fostering care of the men of the North and East, are being developed with wonderful rapidity.

Before the days of the railroads, and up to the time when rail connections were made with St. Louis, New Orleans and Kansas City, Galveston controlled the entire commerce of the State; purchasing all that was sold, selling all that was bought, and levying tribute on nearly every business transaction within its borders. Little did it matter that the gateway to the sea was blockaded by sand bars—which exacted an additional tribute from the producers and consumers of the State to cover the cost of the lighterage charges, for there was no competition to divert the traffic; prices generally were high, and there was enough for all and to spare.

These were the days which made for Galveston the nineteen millionaires attributed to this little city of thirty thousand people in the New York *Tribune's* list of American millionaires.

But the conditions changed as the bonds were broken, when their richest territory was tapped by the railroads from the northern and eastern cities. Then began a period of active competition for the commerce of the State, which had been so profitable to Galveston. The activity of this competition, and the consequent reduction in profits, have saddened the last years of that earlier generation of Galvestonians which now is rapidly passing away. But their sons have adapted themselves to the new conditions, and are making their presence felt in the Southwestern commercial world. To them belongs the credit of enlist-

* Manuscript received December 7, 1896.—*Secretary, Ass'n of Eng. Socs.*

ing the co-operation of the progressive spirits of the great Western States in a united effort towards interesting the United States Government in the great work of improving the entrance to their beautiful harbor.

The people of Kansas and Colorado, chafing under the alleged extortionate charges of their rail outlets to the Atlantic seaboard, quickly responded to the appeal, and through their representation in Congress induced the Government to undertake the work of removing the bars at the entrance to Galveston harbor.

Along the entire line of the Louisiana and Texas coast, from the mouth of the Mississippi to the Rio Grande, there was nothing to compare with the land-locked harbor of 451 acres comprising Galveston Bay, and there has never been any division of opinion among the United States engineers as to the wisdom of deepening the entrance to this inviting harbor, capable of floating the navies of the entire world.

Between the northeast end of Galveston Island, and what is known as Bolivar Point—a peninsula extending southward from the main land —is the principal outlet for the waters of Galveston Bay. It is about three miles across from shore to shore, and through this channel ebb and flow the tides from the Gulf of Mexico, which, meeting the littoral currents which flow generally in a southwestern direction along the coast, deposit the sands which they carry and form what is known as the outer bar to Galveston Harbor; it is about four and one-half miles out from the shore. This it is which interfered so seriously with navigation.

The $77,000 expended prior to 1874 was wasted in a fruitless effort to deepen the channel by means of dredging, and when operations of this character were finally suspended, the natural depth of 12 feet over the outer bar at mean low tide and 13 feet over the inner bar had not been increased.

Then it was—in January, 1874—that the Board of Army Engineers adopted the famous Gabionade project, which contemplated the construction of two gabionades, or training walls, for the ebb and flood tides; the one, from Bolivar Point seaward, on the north side of the channel to the outer bar, and the other from the extreme northeasterly end of Galveston Island along the south side of the channel, in a direction generally parallel with the north gabionade, extending out from the mainland at Bolivar Point.

Since the failure of the gabionade project these sea walls have been known as the north and south jetties, respectively.

The Board of Engineers seemed to have been in doubt as to the proper height, distance apart and length of these two gabionades. They were to be training-walls for the under-currents which they theorized were the cutting agents on which they must depend for the deepening of the channel, but were not to materially interfere with the extraordinary ebb floods which at times occur in this region with such disas-

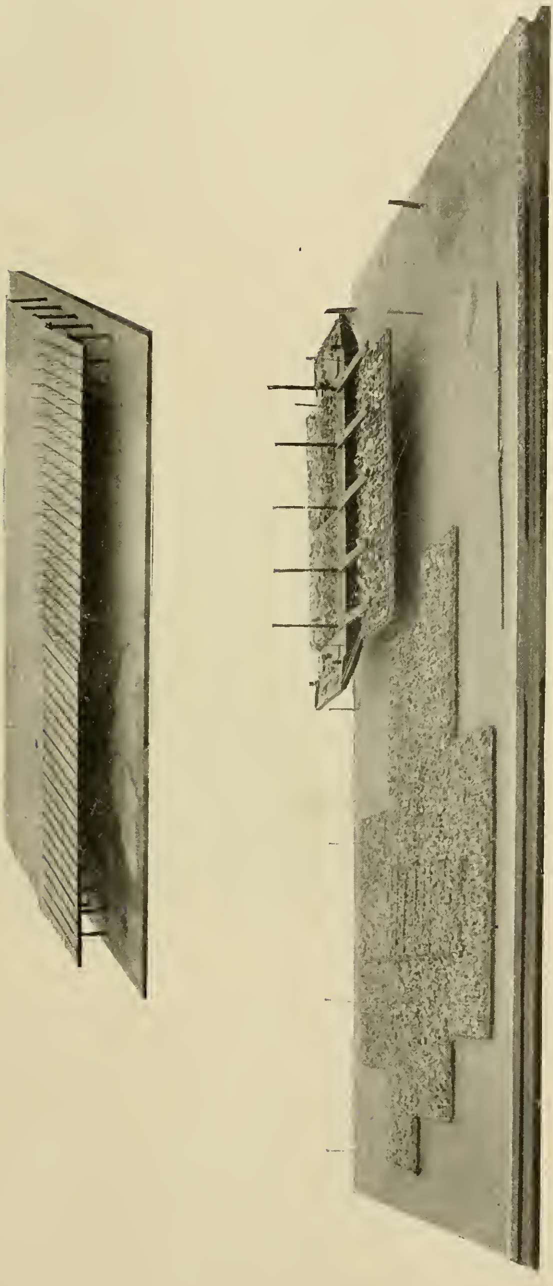


FIG. 1. MODELS FOR WORLD'S FAIR, SHOWING MATTRESS OPERATIONS.

trous consequences to the improvements on Galveston Island, and cause the waters of the bay and the waters of the Gulf of Mexico to meet in the streets of the city. In the storm of 1875 the general level of the sea at Bolivar Point was $8\frac{1}{10}$ feet above mean low tide, while the waters were said to have been banked up at the upper end of the bay to a height of 12 to 15 feet, caused by a stiff ten-days' blow from seaward. The damage was done when the wind shifted suddenly to the north and blew a gale, driving the waters over and around the island into the gulf.

These training-walls were not necessarily to rise above the surface

FIG. 2. THE SOUTH JETTY OF SANDSTONE.

to accomplish the purpose for which they were intended. They were expected more especially to arrest the sands which are carried along the coast by the littoral currents, and to prevent the deposits in the line of navigation. The area of cross-section between Bolivar Point and the end of Galveston Island was 155,611 square feet. The hydraulic mean depth was $17\frac{3}{10}$ feet, and the maximum depth was 42 feet. The mean daily tide at Bolivar Point is one and one tenth feet.

Each jetty-wall was to consist of two rows of gabions, parallel and adjoining, and laid on willow mats resting on the sand. The mats were placed in position by divers and weighed down with concrete blocks

weighing about two hundred and thirty pounds each. The gabionade was built between two rows of guide piling, and the filling with sand was done after the gabions were placed in position. A gabion was nothing more nor less than a woven pine stake-box, 6′ x 6′ x 12′, coated with cement inside and out and filled with sand, using larger size gabions for the deeper water.

The work done during the first season was largely destroyed by the great storm of 1875, and the gabions were badly broken and displaced. The Board of Army Engineers again convened to decide whether the gabionade system was a failure or not. The records show that after

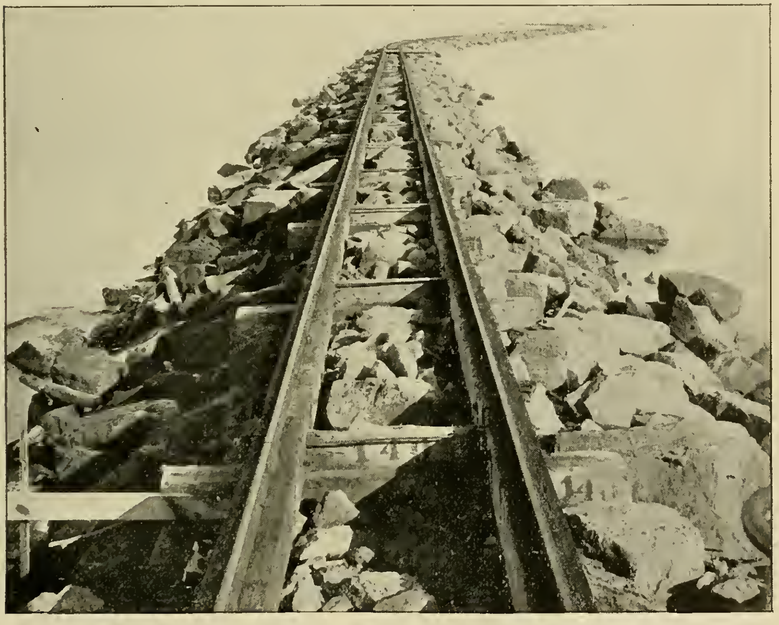


FIG. 3. THE SOUTH JETTY, 14,000 FEET FROM SHORE.

making minor modifications in the plans it was decided that the project should be carried on, and that it was so done until 1879, when the Board of Engineers convened once more for the purpose of passing upon the results attained after 7,332 feet of the outer end of the north jetty had been built and some considerable portion of the inner ends of both north and south jetties. It was decided that, so far as the outer bar was concerned, the gabion plan was a failure, as no satisfactory results had been accomplished. The channel over the outer bar was no deeper; the moving sands carried by the littoral currents had not been arrested, and the gabionade itself was in very bad condition.

So (November 28, 1879) operations on the gabionade plan were permanently suspended, after having built 9,606 feet of gabionade in the north jetty and a considerably less amount in the south jetty.

At this time we wonder why the engineers should have expected an increased depth over the bar without a training-wall on both sides of the channel, especially when the depth on the inner bar, where the waters had been confined, had increased from 13 feet up to 20 feet. It is likely, however, that the real reason for suspending operations on the gabionade was because of the unstable and unsubstantial character of the gabions themselves. It seemed impossible to keep them in position;

Fig. 4. The South Jetty; A Finished Section.
Sandstone Core-Granite Cover.

the storms broke them open and washed out the sand filling, demonstrating the unwisdom of battling with the seas with structures so frail in construction. One curious feature of this experimental work was the fact that the gabions resting on mats seemed to sink into the sand as rapidly as those without, the settling of the gabions ranging from 3 to 7 feet. Another unpleasant surprise was caused by the failure of the gabionade to cover itself with the drifting sands, as was fully expected, and which, in fact, was the vital feature of the gabionade project.

During the entire period of the writer's operations in Galveston Harbor, in 1892 and 1893, he failed to observe a single trace of the existence of any portion of these famous gabionades; and were it not for the statements of history and the Government's increased charges against the improvement from $70,000 in 1874, when dredging operations were abandoned, up to $600,000 in 1879, when the gabionade project was permanently abandoned, one would hardly believe that such operations had ever been carried on there.

Once more the Board of Engineers convened, and the willow-mattress project was the result. The Colonel of Engineers in charge

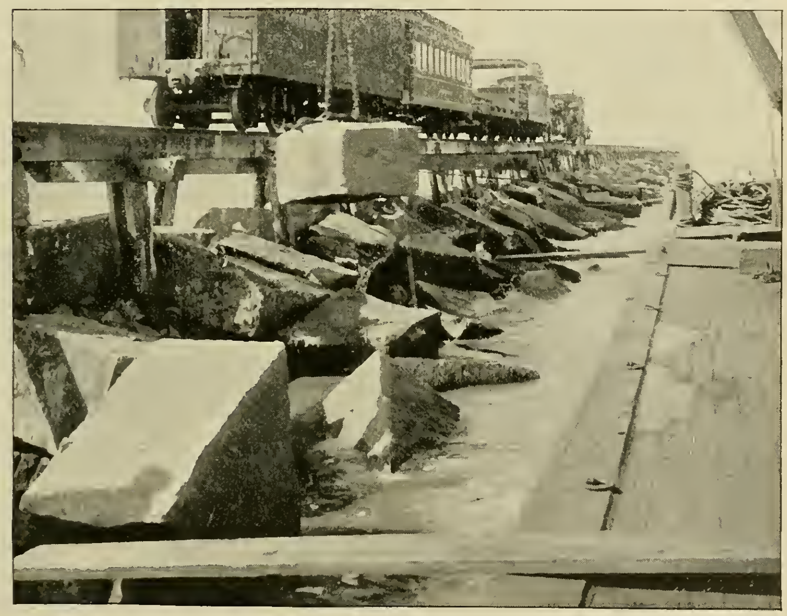


FIG. 5. THE SOUTH JETTY. PLACING THE GRANITE BLOCKS.

said in his report: "It is now intended to build the jetties of brush and stone on a system which will undoubtedly succeed, for it has been applied to open sea exposure at the mouth of the Maas, where it has realized all anticipations and established a certain and economical way of constructing these sea works on sand coasts."

The top of the jetty was to be five feet below the level of mean low tide; twelve feet wide for first 4,080 feet; then gradually sloping up to the surface, with the top width increasing up to twenty-four feet, at a distance of 10,220 feet from the shore; from thence on with a uniform width of twenty-four feet on top to a point 14,960 feet from the shore, where

began the end slope, 370 feet long, downward to the bottom. The gabionade project was figured to secure 18 feet of water; the mattress project 25 feet, and the later project, which is the one being carried on to-day, is expected to produce a channel of 30 feet.

As soon as the mattress project was decided upon, Congress was asked for an appropriation of $500,000, and operations under the new project were vigorously carried on until 1886; but the work was confined to the south jetty, which was practically completed to the outer edge of the bar. The north jetty was left untouched and, naturally enough, there was no increased depth of water on the bars by reason of the operations during

Fig. 6. The South Jetty. Five Derricks at Work Placing Granite Cover.

these six years, although a large amount of money had been expended, including a donation of $100,000 from the City of Galveston.

At this time it was noticed that the mattress jetty was sinking and disappearing, and a line of levels run over the top of it showed a profile ragged and broken. Closer investigation revealed the fact that the rapacious sea worm—the *Teredo Navalis*—was rapidly eating up the brush-work of the entire jetty. The danger of this seemed never to have been thought of. In 1892 and 1893, like the famous gabionades, there was little to indicate that such a thing as a completed mattress jetty had ever existed.

Again the Board of Engineers convened, and the present project, consisting of two practically parallel solid-rock jetties, was adopted. The surface of the jetties was to rise to a height of 5 feet above mean low tide, to be 12 feet wide on top, with natural slopes. The outer ends of the finished jetties were to be in 30 feet of water and 7,000 feet apart. The general lines of the gabionade and mattress jetties were to be followed. Excepting near the shore end of the south jetty—where the wall consisted exclusively of sandstone—the core of the jetty was to be of sandstone and the covering of top and slopes of granite blocks, increasing gradually in size from three-quarters of a ton at the shore end up to

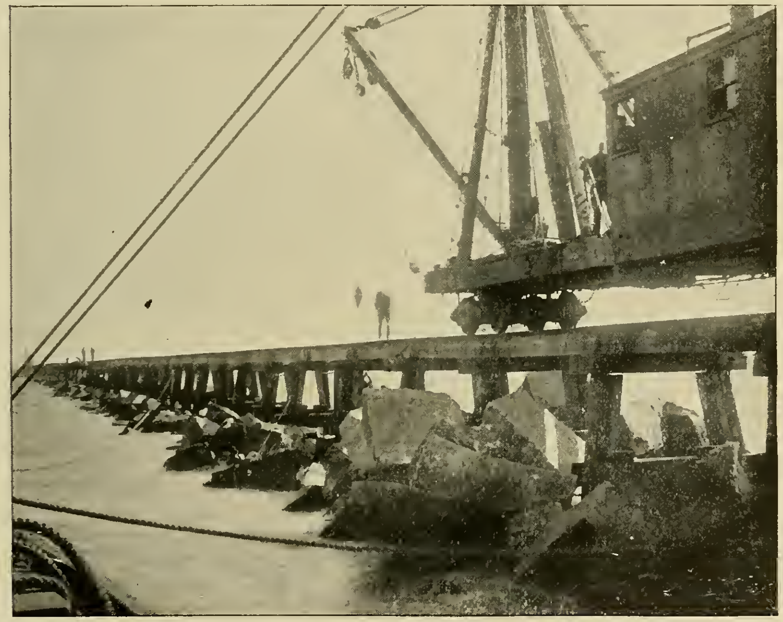


Fig. 7. The Extreme End of South Jetty in 1893. Pile Driver and Car Derrick at Work.

not less than ten tons, nor more than fifteen tons at the outer end. Sandstone was specified to weigh not less than 140 pounds to the cubic foot.

In accordance with these plans, the work of jetty building was again begun, and for ten years has been carried on at a speed depending upon the liberality of the appropriations. At first the various portions of the work were handled by separate contracts, the extent of which was limited by the appropriations; but later the improvement was placed on the Sundry Civil list and a contract for the entire work was authorized

and made. Under this contract the work for a number of years has been carried on at a cost of about one million dollars per annum. On June 30, 1895, the Chief of Engineers reported the total charges against Galveston Harbor as aggregating $5,590,970.40, and the estimate of amount needed to complete the project to be $1,700,000, making a total of nearly $7,300,000. Under active competition at the time of the letting of the present contract, the Government secured low prices, as follows, viz., $2.35 per ton for sandstone in place in the jetty wall, and $4.10 to $4.40 per ton (depending on the size) for the granite blocks. Sandstone had to be hauled 150 miles and the granite about 250 miles.

In the prosecution of work under this contract, the successful contractors early discovered a very serious mistake they had made of guaranteeing to furnish sandstone weighing 140 pounds to the cubic foot, when much of the Texas sandstone weighed but 128 pounds. It developed that it was practically impossible to carry out the contract. A mutual concession of ten pounds in weight for ten cents in price to a large extent overcame the difficulties at a sacrifice to the contractor of practically all the profit on the sandstone portion of the work.

At the end of the fiscal year, June 30, 1895, the south jetty had been extended seaward 32,829 feet, with all but 829 feet entirely completed and 5,100 feet of additional extension yet to make, while the north jetty had been built 22,500 feet seaward, with 4,000 feet incomplete. It will be noticed that the south jetty is about 10,000 feet longer than the north jetty. The most of this difference is due to a long shore connection over the low-lands at the extreme northeast end of Galveston Island, and running in a general direction parallel with the coast instead of out to seaward.

As a direct result of these operations, the depth of water on the outer bar was increased from 12 feet—the natural depth—to 14½ feet June 30, 1894, and to 17¾ feet June 30, 1895. Unofficially, the writer learns that it is now about 20 feet at mean low tide.

Meanwhile, the inner bar has practically disappeared, there being 24½ feet of water there June 30, 1894.

Although the jetties were gradually increasing the depth on the outer bar, it was decided some two years ago to supplement their work by dredging operations with a very powerful suction dredge, and a boat was built by the Government expressly for this purpose, and has since been put to work with very satisfactory results, as reported. Meanwhile, the jetty-walls are being pushed seaward to the 30-foot contour line, where they will stop. Whether the 30-foot contour will move further out with the diversion of the littoral currents to seaward, remains to be seen. The writer is inclined to believe that such will be the result, and that the jetties must eventually be extended still farther seaward, though the movement will likely be very slow.

Although there has been some settling of the stone jetties already constructed, necessitating moderate repairs, yet there is every evidence that the fourth project formally adopted by the Board of Army Engineers will be a pronounced success, even if the first three were complete and costly failures. It will provide at least 25—and with the aid of the powerful dredge-boat probably 30—feet of water where naturally there was but 12 feet over the outer bar.

The cost of the work will have been excessive, due to the enormous amount of experimental work. But the advantages resulting to the people of Kansas, Colorado, Oklahoma, Indian Territory, Arkansas and Texas of deep water at Galveston will many times repay even this excessive cost.

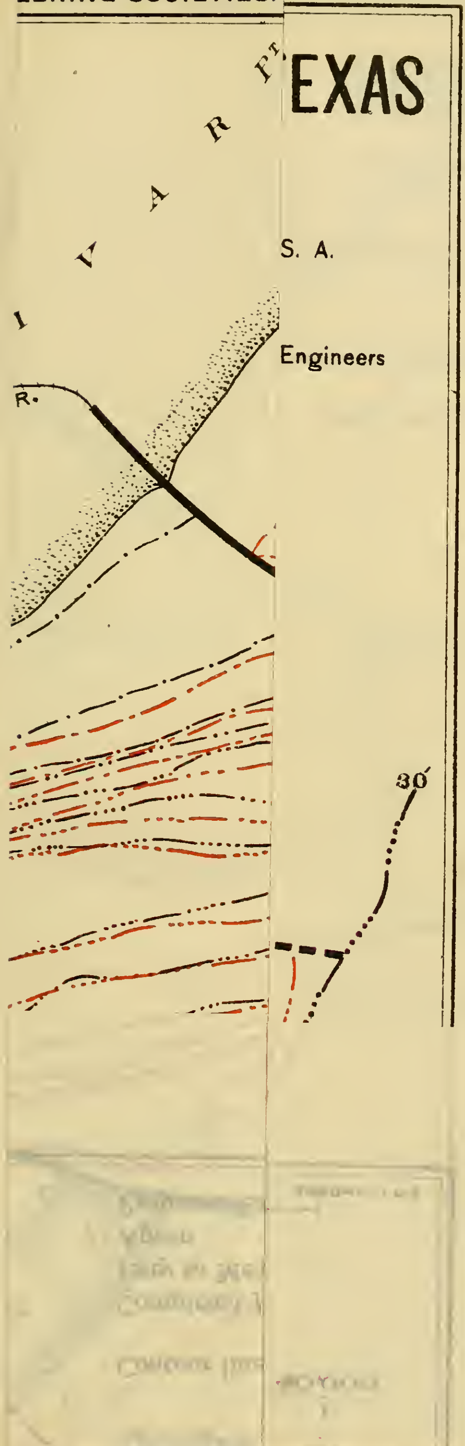
The commerce of the port of Galveston has increased from 863,196 tons in and out in 1877 to 1,211,354 tons in 1882 and 1,452,832 tons in 1895.

You, who were at the World's Fair, will perhaps remember the models exhibited there, illustrating the methods of the third or brush-mattress project. In Fig. 1 you have a view of these models, showing in a general way how these mattresses were built, floated to position, sunk into place on the sea bottom and weighted down with broken stone. The shore end of the south jetty appears in the back ground. Fig. 2 is a view of the south jetty at a point 9,000 feet from the island, and where the deflection to seaward is made. Fig. 3 gives a good view of the awkward curves introduced into the south jetty, and shows plainly the method of constructing the railway built on this jetty wall for the purpose of delivering materials. Figs. 1 and 3 show that portion of the south jetty, which is built entirely of sandstone.

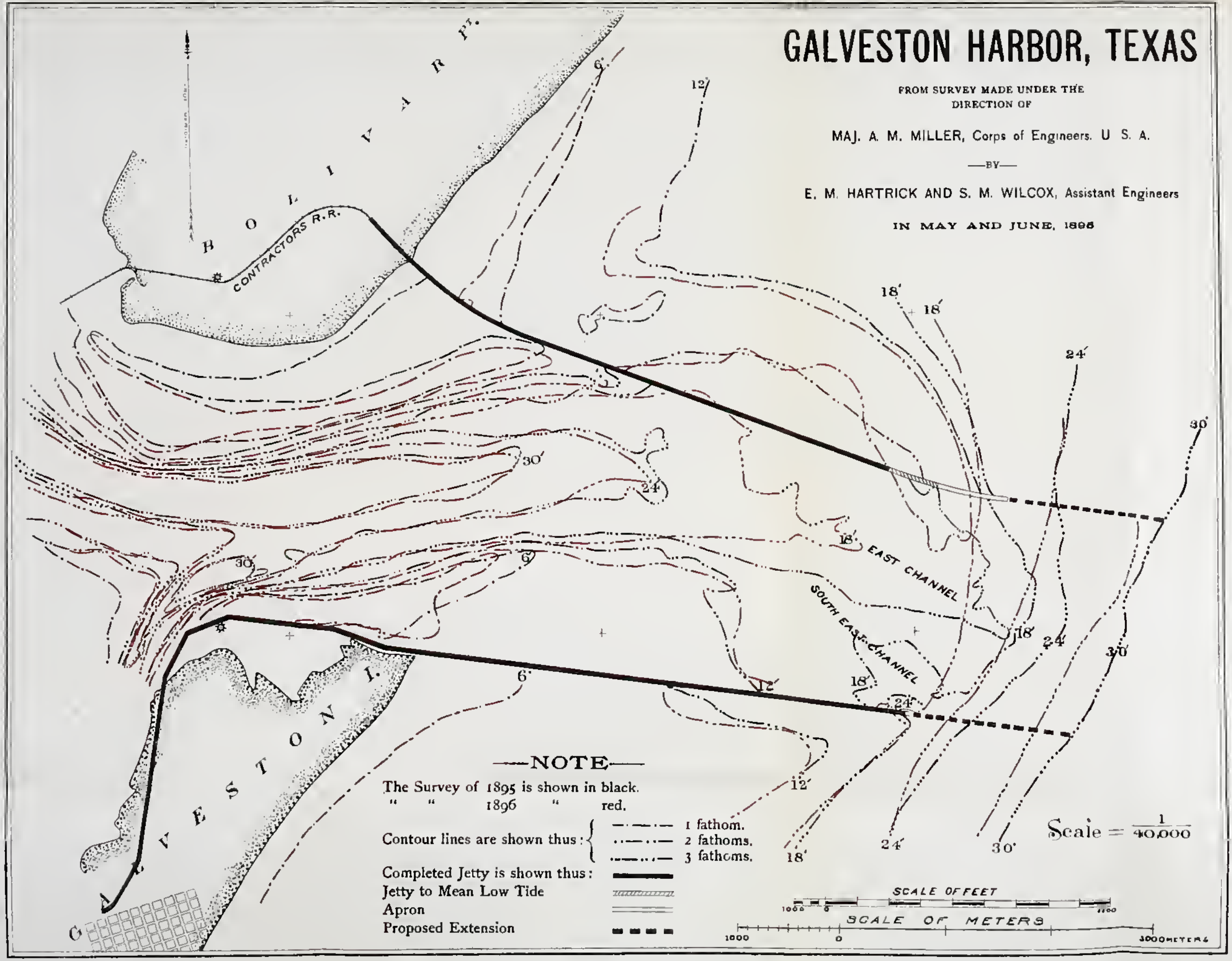
Fig. 4 shows the finished jetty and the granite blocks after they have been placed in position with a derrick. Fig. 5 represents one of the large derricks at work placing a block of granite, with a train on the trestle in the background. Fig. 6 is a view of four derricks on the two barges and a track derrick, all at work placing granite blocks and putting the finishing touches on the jetty. Fig. 7 shows the granite blocks piled roughly on the jetty, after being unloaded from the flat cars which brought them from the quarry, and also the end of the jetty with the piledriver, driving two piles in a bent in the distance, advancing the trestle work ahead of the jetty, and the car derrick at work in the foreground. Fig. 8 is a general map of the entrance to Galveston Harbor.

The work on the north jetty was begun and carried on for some time in practically the same manner. There being no railroad on Bolivar Peninsula at that time, a car ferry was established and the trains of rock brought over from the railway yards of Galveston. This method,

however, proving slow and expensive, it was supplemented by a barge line, which was established to a point on the mainland, near Houston, and deliveries of rock made directly into the jetty. With the aid of the barge line the work has progressed rapidly, and if the Government continues the appropriations at the rate of a million dollars per annum, we may hope ere long to read the announcement that this mammoth undertaking is finally completed, and that the Port of Galveston can be entered by any ship which floats.



Manuscript received December 14, 1896.—*Secretary, Ass'n of Eng. Socs.*


GALVESTON HARBOR, TEXAS
FROM SURVEY MADE UNDER THE DIRECTION OF
MAJ. A. M. MILLER, Corps of Engineers. U. S. A.
—BY—
E. M. HARTRICK AND S. M. WILCOX, Assistant Engineers
IN MAY AND JUNE, 1896
BOLIVAR PT.
CONTRACTORS R.R.
GALVESTON I.
EAST CHANNEL
SOUTH EAST CHANNEL
6'
12'
18'
24'
30'
—NOTE—
The Survey of 1895 is shown in black.
" " 1896 " red.
Contour lines are shown thus: 1 fathom. 2 fathoms. 3 fathoms.
Completed Jetty is shown thus:
Jetty to Mean Low Tide
Apron
Proposed Extension
Scale = 1/40,000
SCALE OF FEET
SCALE OF METERS
1000
0
3000 METERS

STRUCTURAL STRENGTH OF SHIPS AND IMPROVED ARRANGEMENTS FOR REPAIRING THEM WITHOUT DIMINUTION OF THEIR STRENGTH.

By Joseph R. Oldham, N. A. and M. E., Member of the Civil Engineers' Club of Cleveland.

[Read before the Club, November 10, 1896.*]

In the commercial world it appears to have become an established conviction that dividends cannot now be increased by leaps and bounds, as was the case some years ago; consequently the merchants, manufacturers and shipowners of to-day seem to concentrate their energies upon the perfecting of details connected with the handling or transportation of their merchandise, upon improvements of their plant, or upon a reduction in certain items of expenditure of which, but a few years since, only very casual notice was taken. With shipowners, the tendency is to largely increase the capacity of vessels so as to carry greater dead-weight in units of displacement and with but a small increase in working expenses. The science and practice of engineering have become so perfect and their secrets so widely known, that no very large advantage can be secured by one efficient and painstaking engineer over another through any radical departure in design or in the general system of construction applicable to similar structures. Our *marine* engineers and shipbuilders are similarly circumstanced, and those who are actuated by the laudable desire to be second to none, find their safest course towards pre-eminence in the careful study of details of construction, by which means a sure though small advantage, slowly acquired, may be secured over less thoughtful and less methodical competitors.

The first iron steamboat ever built was launched in the year 1821. The oldest steamer now in active service in the world, however, is afloat on lake Erie, the United States Steamer "Michigan," which was built at the city of Erie on this lake, in the year 1844. That was before the celebrated "Great Britain" had crossed the ocean, and in the same year that the first Cunard steamship made her initial trip to Boston.

During the last fiscal year 108,782 tons of shipping was built on these lakes. Such an output of steel tonnage has never before been made on the banks of the unsalted seas. Indeed, this is nearly equal to two-thirds of the tonnage constructed on the River Wear during the last nine months, and I may remind you that Sunderland is the greatest shipbuilding port in the world. This year's output there will be fully 230,000 tons.

*Manuscript received December 14, 1896.—*Secretary, Ass'n of Eng. Socs.*

Our steel tonnage has been increased fully 40 per cent. since 1893. The capacity of the vessels built on these lakes this year is 86 per cent. larger than that of those built three years ago. On the ocean, however, vessels have recently been constructed to carry 13,000 gross tons.

STRENGTH OF SHIPS.

Professor Mosely says that the strongest form that can be given to a solid body, in the formation of which a given quantity of material is to be used and to which the strain is to be applied under given circumstances, is that form which renders it equally liable to rupture at every point, so that when, by increasing the strain to its utmost limit the solid is brought into a state bordering upon rupture at any one point, it may be in the state bordering upon rupture at every other point. Moreover, the strongest form is also the form securing the greatest economy of material.

As to the structural strength of ships. It is not unusual to find strain existing in the side plating butts below the upper deck and in the bilge plating butts above the bottom while the upper shear strakes, deck stringer plates, keel and bottom plates show no signs of distress. Now such straining could not be caused by longitudinal bending, because stress due to bending moment is a maximum on the upper stringer and shear strakes and at the keel and lower bottom plating alternately. The distress at the upper turn of the bilge might be attributed to transverse bending as the break in the framing above the upper bottom is an element of transverse weakness, but there is no such discontinuity of strength between the upper deck and the side plating. From this it appears that such signs of straining as those just indicated could not be caused by longitudinal bending alone. When it is understood, however, that longitudinal shearing stress generates equal shearing stress in a transverse direction, the distress frequently observable on the sides well above the bottom and near the neutral axis, and below the upper decks, in ships having great longitudinal strength, may not be so difficult to explain. The late Prof. Jenkins pointed out that in the case of a body subject to bending moment as well as to shearing stress, the distribution of shearing stress differs materially from that in the case of a body subject to shearing stress only. In this he is in accord with Rankine, who showed that a shearing stress, when combined with a bending stress, is not uniform over the section, but is greatest at the neutral plane and least at the top and bottom. The excess of the maximum shearing stress over the mean depends upon the arrangement of the material in the section. On the contrary, the longitudinal bending moment is a maximum at the top and bottom and nil at the neutral axis. If the truth of these statements requires confirmation, such may

be found in the fact that in steel shafts, disintegration begins at the center, and gradually spreads until it reaches the surface, when fracture occurs without warning. Of course it will not be assumed from this that the structure of a floating body is free from stress along the neutral plane, for such cannot be the case, as besides a vertical longitudinal, there is also a horizontal transverse bending moment, which produces maximum stresses at the longitudinal neutral plane. But of more importance than this is the stress due to shearing moment, which, as I have said, is maximum where stress due to longitudinal bending moment is nil. In addition, the force of the waves has to be resisted at all parts of the external surface of the hull.

Sir William Fairbairn established the practice of the mathematical investigation into the strength of a ship considered as a hollow girder so far as longitudinal bending moment is concerned. The principle is the same as that by which the strength of a beam may be calculated.

It follows, therefore, that the sum of the products of the small elements composing the section of a ship, such as keel plates, stringer plates and the effective area of steel hatches, multiplied by the squares of their respective distances from the neutral axis, will constitute the moment of strength of the entire section. The principles generally governing the strength of beams or girders enable us to compare the relative importance of any assemblage of plates and bars such as are commonly selected to form the structure of a modern steel ship.

STRAINING OF SHIPS.

Mr. Fairbairn assumed that ships, whether afloat or grounded on a rock, are governed by the same laws of strain as simple-built beams, such as tubular bridges. Under certain test conditions he investigated the longitudinal strength of an actual vessel, and advocated plating over the upper deck beams, so as to make the section of iron at the deck equal to the section at the bottom. If this could be attained one of the greatest troubles of the naval architect would cease to exist; but, as the requirements of loading and unloading lake steamers now are, the shipbuilder has a complicated task on his hands. Not only are the hatchways required to be extremely large, but greater loads are demanded on almost the same draft of water as existed when cargoes of half the weight now carried were deemed satisfactory. If we consider a ship in her upright position when afloat or poised on the rocks with the water partially ebbed away, she may be looked upon as a hollow girder, and as such, if it were possible, her transverse mid-section would by preference be formed with a solid deck, making the top and bottom flanges equal. But the actual conditions require the solid deck

to be reduced for hatchways. Now, without doubling the thickness of the upper deck side plating and otherwise adding strength to make up for the large portion of the decks cut away for the hatchways, and amounting to about 65 per cent. of the total breadth of the deck plating, the longitudinal strength of the symmetrical girder, as compared with the perforated girder, is as 600 is to 325. A ship may in some respects be looked upon as a loaded girder, but the stresses to which she is subject are much more complicated. A vertical flange in a bridge remains vertical, but a vertical flange in a ship on the stocks may become horizontal, or nearly so, when she is working in a seaway. Moreover, one side of a vessel may be in tension above water at one moment and be in compression under water at the next moment. May not this account for the straining of butts in the locality of the neutral axis, and for the fracture of plates at a considerable distance from the gunwale or top flange?

MOMENT OF INERTIA.

I have calculated the moment of inertia of a modern steel lake steamer at her midship transverse section. I also show the stress per square inch to which she would be subject without steel hatches. You will observe that with the latter her neutral plane is greatly raised. This is very desirable, for the average lake steamer has her neutral axis much too low. I will proceed on the assumption that the longitudinal bending moment of a ship afloat is greatest when crossing waves of her own length, and at the instant when the crest is amidships, the weight and buoyancy being equal. The principal dimensions of the steamer investigated are:

Length on water line	400	feet.
Breadth moulded	48	"
Depth, total without hatches (as a girder)	27.8	"
" " with steel hatches	29.1	"
Displacement at 17 feet mean draft of water . . .	= 7,400	gross tons.
Half moment of inertia	= 65,034	

As the weight and distribution of cargo in our steamers is so varied it would hardly be justifiable in this instance to spend the time required to calculate the exact bending moment with an ideal cargo. So, in order to obtain this, I will take a coefficient as a fraction of the weight (W tons) into the length (L feet) to obtain the bending moment. Annexed will be found the divisor for bending moments of seven steamers. Maximum bending moment $= W \times L$ divided by one of the following numbers:

Hogging on wave crest.	Sagging in wave hollow.
24.4	117.
29.9	91.8
37.	83.
37.	43.
36.	50.6
37.6	39.7
27.8	55.7
32.79 = mean.	68.68 = mean.

The following is the moment of a well-deck tramp, 290 $\times$ 38 and 3,590 tons displacement: $\frac{3{,}590 \times 290}{80} = 13{,}000$. From this it appears that her maximum bending moment equals 13,000 foot-tons. As lake steamers are more severely loaded and float much closer to the bottom than ocean vessels, I use 50 as the divisor for ascertaining the bending moment * Then the coefficient of the steamer illustrated $= \frac{59{,}200 \times 18.78}{130{,}068} = 8.54$ tons tension per square inch of section at gunwale, and $\frac{59{,}200 \times 10.32}{130{,}068} = 4.62$ tons compression on the bottom. These are the stresses with steel hatches. Without steel hatches the top sides are much more severely stressed. Thus $\frac{59{,}200 \times 19.7}{121{,}604.2} = 9.332$ and $\frac{59{,}200 \times 9.33}{121{,}604.2} = 4.542$ tons per square inch on bottom. Without hatches the neutral plane is 19.17 feet below the gunwale. With steel hatches this distance is reduced to 18.78, and this causes the discrepancy between the stress on top and bottom of the girder to be largely reduced.

The moment of inertia, based on the transverse mid-section and the bending moment due to the maximum load when the vessel is afloat in the largest seaway, should be calculated for every new design.

IMPROVED HATCHES.

I can see no necessity for increasing the strength of the bottom; for, as ships are generally constructed with double bottoms, the bottom is the strongest of the four cardinal surfaces; and, in most ships, the top or deck is the weakest. Therefore my present efforts with regard to structural strength have been concentrated upon the upper deck, with a view to compensate for the immense openings cut therein for hatchways. The common oak hatches or "hatch covers," as they are sometimes called, do not contribute in the slightest degree to resist the tensile stress due to hogging moments; but the steel hatches I have designed, even though very thin, will largely resist the stresses resulting from either hogging or sagging forces.

I will ask you to permit a digression while I describe certain anomalies connected with naval architecture, and which I am led to believe are not confined to ships alone, but may be discovered among the exceptions to be found in other large mechanical structures. I refer to flimsy examples of naval architecture which some of us may have sailed on, with or without the knowledge of the fact. As you are aware, most modern ships are constructed more or less in accordance with certain

* Bending moment at mid length $= \frac{7400 \times 400}{50} = 59{,}200$ foot-tons.

rules formulated and tabulated by the great ship classification societies. Some of these rules are very valuable and reliable, and, when faithfully adhered to, they will produce a strong and seaworthy ship, but, even then, the production will, I fear, fall short of that ideal structure, the dream of the shipbuilder and the vision of the shipowner, wherein the maximum of strength will be found closely associated with the minimum of weight.

USEFUL WEAK SHIPS.

The above title may sound paradoxical, and indeed the pre-eminence of such vessels as dead-weight carriers does not exactly result from their weakness, but from the contributing cause, viz: their lightness or lack of scantlings. Permit me to say however, that a weak structure may not be light and a light structure may not be weak. Such ships as I refer to—and I can call to mind several of them—are by no means peculiar to these lakes nor are they confined to any nationality. Every great maritime nation has one or more of them. They are generally the product of economical experiments unconsciously made. They were built regardless of now well-known scientific principles and reached their present stage only after many years of trying to do without something. Those useful ships, such as the "Michigan," the "John Bowes," or the "Tiber," which are afloat to-day, represent the "survival of the fittest." For instance, where the text-books say that $\frac{10}{16}$ is required, they have only $\frac{8}{16}$; where the rules require triple or double riveted butts, these flimsy structures have only two rows, and, in many cases, only a single row of rivets; though their scantlings correspond to easy proportions, their ratio of length to depth is comparatively extreme. In fact, they were built according to that slow but sometimes sure practice known as the rule of thumb. A word about this rule—if I may give it such a title. I have known men, and many of you know them too, who were born engineers or shipbuilders, and, instead of calling their guide the "rule of thumb," I would say that they act from an experimental knowledge, which became reliable as they became exact, which became comprehensive as they became experienced, and which became more useful as they became more cautious.

It must not be supposed from this that I undervalue the study of scientific works, or that I think lightly of text-books or books of rules; by no means. A man's practical experience, even though he be a great genius, such as Robert Stephenson or James Watt, is necessarily limited, and in the work he may do (though it be very useful and valuable), it can guide him only to a very limited extent. His knowledge may be intuitive, but his designs must be circumscribed and imitative. Only a scientific engineer could have projected and designed a "Great Eastern," which was ten times as large as any of her predecessors.

A safe Tay bridge could not have been constructed with anything like its symmetry and lightness without the aid of scientific knowledge and formulæ. The point I desire to make here is only this—that when we come to a complicated composite structure, such as a large steamship, our books do not show us how to make them with a certain amount of material so that any one part will be as liable to rupture as any other; but this is sometimes accomplished by unscientific mechanics after a long experience in constructing works of similar design. For instance; in our ships, constructed according to modern rules, the upper deck stringer plates and angle-bars have sometime no factor of safety, while other parts, such as the bottom and portions of the sides, have a factor of five or six. Our boiler shells have a factor of about six, the furnaces and fire-boxes frequently have no factor. So, with many of our modern ships, there is an excessive factor in many places in spite of our boasted knowledge, whilst some modest, practical old shipbuilders have floating monuments on the great seas to-day which prove by their existence that they have a factor of safety, and by their dead-weight that there is in them little or no waste of material. They are beautifully simple; and, as you know, simplicity, in all things, but proverbially in mechanics, is supreme excellence. This should indicate that some of the elements composing the structure of a ship are heavier than necessary, and, notwithstanding years of experience—the results of which are tabulated from innumerable examples of carefully-designed ships—it is quite evident that there is a large redundancy of strength in many parts of our modern steamers. Of course, there would be but little loss in such cases were it not for the extra weight and cost invariably associated with such local excess.

When a steamer is heavier either in hull or equipment than the work she has to do warrants, such superfluous weight will handicap her as long as she floats.

To bring this nearer home, I may say that there are old steamers afloat on these lakes and of moderate depth, too, the top-sides and upper decks of which are so poorly connected and largely perforated as to make a shipbuilder wonder how they ever got across Lake Erie, whereas they have not only crossed this little lake, but have done good work over all these dark and stormy waters for more than twenty years. But these vessels have main decks as well as light upper decks, and on that point I would like to say a few words. It has been stated, and the statement appears for the immediate present to have been confirmed by practice, that a main deck is useless unless required for carrying cargo. Now, though I readily admit that one of its greatest uses in a broad, shallow steamer is to support such weight, it is also of special value, at least in an ore-carrier with its weak upper deck, to strengthen the hull horizon-

tally and transversely, though it may not contribute largely to its strength in resisting bending moment. Moreover, a good main deck not only strengthens the beams by forming a broad top flange, but it also relieves the beams of a great portion of their work, which is to resist panting. Though such a deck may not always be required to carry cargo, the strength of a second deck is invariably required to support the sides of long vessels when the depth of hold is about twenty feet or over. Of course, beams can be made so strong and so well braced that a main deck may be dispensed with. Such beams, however, should partake of the box-girder form; but then, with the weight of stringers and ties, this arrangement would approximate to the weight of an average main deck.

OCEAN AND LAKE VESSELS.

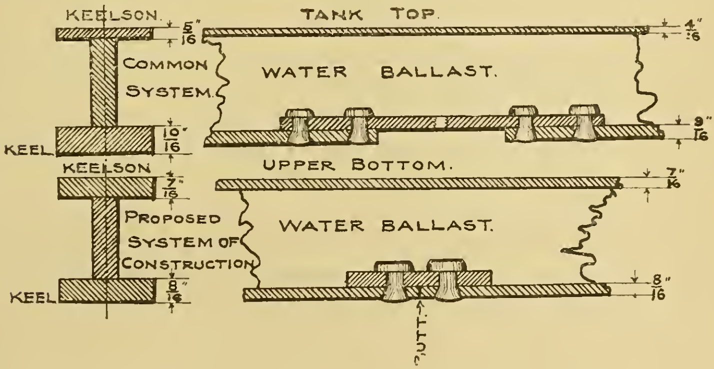
Speaking generally and comparatively, I would say that, while the bottom of ocean steamers should be made for floating, the bottom of lake vessels should be designed for grounding. The practice of lapping plates, instead of butting them, has somewhat recently become popular on these lakes. The system is by no means new, however, as Messrs. Harland & Wolf, of Belfast, adopted it about ten years ago. But, long before they had thought of this means of doing away with the necessity of calking and recalking bottoms and bilge butts to destruction, Mr. John Reid, of Port Glasgow, carried this system into practical effect on the old "Tiber," built about forty years ago, and she was as good an old vessel as I ever examined. But lap joints are not so good, according to my experience and judgment, for the bottom of lake steamers or for the sheer strakes and stringer plates of large vessels. No riveted joint can be stronger than a properly designed double strapped butt joint, though such joints are expensive.

When the strakes of plating are lapped over each other, packing or liners are necessary in order to make close and solid work with continuous fair transverse frames. The necessity of such liners is a serious and still common defect. Messrs. Wm. Doxford & Sons, of Sunderland, England, propose to avoid this by a well-known process called joggling. This innovation in modern ship construction is evidently appreciated by many shipbuilders, and it may be wise to adopt it in the construction of deep-water vessels, which seldom take the ground; but, for our lake trade, I think such a departure would only intensify that defect in ship construction which it seems most desirable to avoid, namely, the system of overlapping inside plates by a portion of outside ones, which makes it necessary to largely loosen and frequently to entirely remove one or more uninjured plates in order to replace an injured one.

Bottom lap joints, on account of the plate there being lower than

the remainder of the plate by its own thickness, receive the greatest stress when the bottom rests on the ground. Plate edges and the calking and riveting of the transverse joints are frequently cut or strained to destruction when the body of the plate is practically uninjured. I, therefore, propose connecting the bottom longitudinally and transversely by butt joints, which give a flush surface, so that all parts of the flat bottom may be equally stressed when the vessel grounds.

As the bottom plating will not be so heavy as bottoms now commonly constructed, I make the ballast tank top much heavier and stronger than is generally the case. For instance, the average thickness of the tank tops of ordinary steamers is about 65 per cent. of the thickness of the bottom plating. This system of construction I virtually transpose, making the bottom thickness no greater than the thickness of the tank top. This tank top is then stiffened and supported by transverse girders of suitable depth and thickness, say about 15



inches deep by $\frac{3}{8}$ inches thick, spaced about two feet apart in a medium-sized lake steamer, and riveted to the tank top in the ordinary manner, except that the fastening thereto will be increased to correspond with the increased thickness of the tank top plates. Underneath these transverse girders I fit longitudinal girders, about one to each strake of bottom plating, say four or five of such lower girders each side of the center vertical keelson plate, which runs from stem to stern or nearly so. These longitudinal girders, which extend down below the transverse or thwartship girders sufficiently deep to provide the proper capacity for water ballast—which will require to be about 40 inches deep in a large lake steamer—will be connected to the transverse girders by suitable fore-and-aft angle bars, or these plates may be flanged, and connected by vertical angle bars, the latter stopping several inches—say four or six—below the tank top or upper bottom, so that the upper bottom plates may not be pierced or seriously injured by the thrust of such

vertical angle bars, when the bottom strikes the ground. These bars may be continued down to the bottom plating or only to the horizontal angle bars which connect the longitudinal girders to the bottom plating. To secure sufficient transverse strength below the tank top, I fit peculiar diaphragm plates between each pair of fore-and-aft girders. These plates will be riveted by means of suitable angle bars or by flanging to the fore-and-aft girders, and to the bottom plating; and they will be comparatively lightly attached to the lower part of the thwartship girders, so that a thrust from the bottom may not fracture these girders. The diaphragm, having an opening or a man-hole in the lower part, will bend or buckle before the tank top could be fractured. Ballast tank tops, however thick they may be, will not long withstand the impact of large masses of ore falling upon them from a height of twenty feet or so without the protection of wood ceiling unless the butts and landings are well stiffened with vertical flanges and a double row of rivets. But with a tank top as thick as I propose, and properly supported, iron ore, coal and such cargoes may be thrown into the hold without fear of injury to the upper bottom. . . . Another reason why the bottoms of lake steamers should not be so thick as the upper bottom, is that groundings are so frequent and bottom plates require renewal so often that such plates are not permitted to remain in the ship sufficiently long to be dangerously weakened by corrosion.

The principal object of my invention is to do away with the lap joints, either longitudinally or transversely, in the bottom plating, for the full length of the cargo holds, and extending from end to end of the midship body of the vessel. Then, after leaving the flat of the bottom, the end plating may be lapped and double-riveted in the ordinary manner. It will be observed that with flush-jointed or edge-to-edge plating, I substitute single for double riveting except in the transverse or butt joints over the midship body; and, so far as tensile or compressive stresses due to longitudinal bending moment are concerned, double-riveted landing edges are not necessary anywhere.

I hope my design has mastered the evil, so far as the bottom is concerned. As regards the sides below the sheer strake, I think that joggling and lapping are all right, for the one tends towards a reduction in weight of hull and the other slightly reduces the cost of construction. I would like to say now that nowhere in the world can ship repairs be effected so quickly and economically as right here on these lakes.

COMPENSATION FOR HATCHWAYS.

In conclusion, let me briefly describe the improvements I propose for augmenting the strength of the upper flange of a ship girder and at the same time increasing the durability and resilience of the hatch covers.

My improved hatch coamings and hatches consist in fitting strong steel or metal hatches or hatchway covers, with angle bars at each side, which will embrace strong T beam or channel bar coamings, so as to increase the longitudinal strength of the decks and upper works, be more durable than the ordinary oak hatches and do away with the necessity for tarpaulins. When the hatchways are small, I make the hatches all in one piece, with the sides flanged down over the top of the coamings; or angle bars may be riveted at the sides, as with larger hatches. By preference, I hinge one side of the hatch to the coamings. The opposite side may then be lifted, by a rope or chain working through a block or pulley connected to the mast or to a post fitted for the purpose, till it reaches such an angle as to rest securely against such post or against a stanchion fixed to the deck; or the hatches may be held open by the chain, hook or catch, or be turned back onto the deck. When the hatchways are large, say over 30 x 8 feet, I make two, three or more pieces and join them by suitable hinges, so that they can be folded up transversely and be drawn to the port or starboard side, leaving the hatch open, or nearly open, according to the requirements of loading or unloading arrangements; or the hatches may be all in one length and be lifted by a suitable crane or by a tackle or chain attached through a block or pulley on the triantic stay at sufficient height for raising the hatches off the hatchways. With hinged hatches, angle or flat bars are fitted on the lower side in a fore-and-aft direction, to stiffen the plates and form a watertight joint when the hatches are closed, by means of rubber gasket or other suitable substance placed between the adjoining flanges; or the hatches may be fitted with suitable tie-rods and stanchions, similar to those commonly seen on the lower sides of railway cars, to keep them in shape and for lifting them off the hatchways and placing them in a convenient position on the decks. Asbestos, rubber or other suitable material may be used to make a perfectly watertight joint between the tops of the hatch coamings and the lower sides of the hatches. The hatches are secured to the coamings by suitable screw bolts and winged or lever nuts, or by common nuts. If the hatches are to be hauled to one side of the vessel, a suitable roller, of such a height as to raise one end of the hatch slightly off the coaming, is fitted in strong brackets to the deck.

A few years ago, tenders were invited for triplicate lake steamers. So far as general design and principal dimensions were concerned, these three vessels were alike, but when I compared them by their displacement, one was found to be eighty tons and another one hundred and fifty tons lighter than her sister ship. This discrepancy represents a maximum loss, in dead-weight ability, of about 7½ per cent. These steamers being constructed under my superintendence, it devolved upon

me to watch their performance for some time, and I may tell you that the results of frequent surveys tended to prove that the lightest of the three steamers was the strongest.

If the shipowners are not too exacting with regard to dead-weight cargo, and if the weights of material are not over-carefully scrutinized in the shipyard, it is by no means a difficult task to design and construct a strong ship.

On the other hand, if the designers are but sufficiently obtuse as to the exigencies of the trade for which the ship is intended, and provided the classification societies are a little accommodating or are not well informed as to the conditions of the work to be done by the vessel, a structurally light, weak ship may as easily be built.

But to produce a symmetrical structure, such as a large modern lake steamer, with a proper factor of safety and with the strength of the material so distributed that no one part of the structure will be more liable to fracture or straining than another, calls for the experienced training of the talented engineer, combined with the precision of the mathematician; for only under such conditions can huge, complicated machines be produced as quickly and economically as our times demand.

A FEW POINTS OF ENGINEERING INTEREST OBSERVED ON A SHORT TRIP ABROAD.

Pavements, Confined Rivers, and the Water Supply of Ancient Rome.*

By Francis W. Blackford, Member of the Montana Society of Civil Engineers.

[Read before the Club, November 14, 1896.†]

The limited time usually allotted to an engineer for travel and recreation often precludes the possibility of close study of engineering works. These conditions confronted the writer of this paper, and he presents to the society simply his observations, with the few details and technicalities that could be hastily gathered.

PAVEMENTS.

Unquestionably American city pavements suffer greatly in comparison with the modern pavements of London and Paris. Upon the busiest streets of both these great cities wooden blocks are in general use. Asphalt is used extensively for the narrower and less important streets, and stone blocks in many places, notably near the wharfs and in the wholesale districts. In the fashionable part of London, the west end, near and in St. James' and Regent's parks, well-kept macadam roads are used extensively.

All pavements are kept in excellent repair, and small gangs of workmen may be seen at almost any time repairing both the wood and the asphalt.

In the two cities mentioned the wooden pavements seem to be most in favor, and, from the best information that could be gathered, they seemed to be growing in favor with all classes, possibly the vehicle manufacturers excepted. By watching the work of repairing the wooden pavements of London and by conversing with the foreman in charge and with the superintendent of the Improved Wood Paving Company that lays and keeps in repair most of them, the following information was obtained: They are composed of blocks of Baltic pine from 6 to 10 inches long, 3 inches wide and 6 inches deep, laid or set upon a foundation of 6 inches of hydraulic cement concrete, the blocks having been first dipped in creosote oil. The top surface of the concrete base is made smooth by a covering or plaster of cement mortar, and after all

* For much of the information here given concerning aqueducts the writer is indebted to Dr. Russell Forbes, of Rome, and to his pamphlet upon the Roman aqueducts and fountains.

† Manuscript received December 15, 1896.—*Secretary, Ass'n of Eng. Socs.*

is thoroughly set, the blocks are placed directly upon the smooth surface of the concrete in rows across the line of traffic, leaving spaces of from $\frac{1}{4}$ to $\frac{3}{8}$ of an inch between the blocks. These spaces are then filled, or run in, with hot bituminous mastic, about one inch and the remainder filled with cement grout. The spacing of the blocks is maintained by three brads driven into the blocks to a shoulder. One row of blocks running lengthwise with the street is omitted at the curb until the swelling of the timber has ceased.

The timber is not thoroughly seasoned, neither is it green, but what is in England called first water timber, being that which is cut in the country tributary to the Baltic Sea during the winter season, shipped as soon as the ice is out of the harbors and probably marketed in about six months after cutting.

The dipping is not a forcing process, by which the creosote oil is forced into the pores, but a simple bath, the blocks not being in the oil more than two or three minutes.

The pavements last about eight years under very heavy traffic, and with lighter traffic much longer. Some that had been in for fifteen years were not in bad condition, and it has been known to last nineteen.

On the approaches to London bridge, where the traffic is the heaviest in the world, estimated at about 400 tons per foot of width per day, it actually wears out in about four years; notwithstanding this the approaches are paved with wood, which of itself is an evidence of its popularity. London bridge proper is paved with stone blocks similar to those used in this country. This is tolerated probably because there are no tenants along the side to be annoyed by the noise.

This character of pavement costs in London about seven shillings six pence ($1.80) per square yard exclusive of foundation, with a probable guaranty for about five years.

The success of this pavement is doubtless due to the care exercised in the selection and treating of the timber, and the careful manner of laying it, together with that requisite of any good pavement, an impervious and unyielding foundation.

On the busy streets of London small boys with shovel and basket gather up the horse droppings during the day and dump them into the sewer through standpipes at the curb line; and at night the entire street is washed down with a hose and scraped with a rubber scraper similar in principle to those used for cleaning plate-glass windows.

At intervals, fine gravel is sprinkled upon the pavement. This is soon driven into the blocks by the traffic, and is thought to increase the life of the pavement, and to better the foothold of the horses. It has the appearance of conglomerate or pudding stone when clean, and would not be recognized as wood unless closely examined. The omnibus traffic,

being both quick and heavy, is thought to be the most destructive to these pavements.

The popularity of this pavement is due not to its cheapness or its durability, but to its noiselessness. The din of the traffic of the Strand and other busy streets upon a cobble stone or well-worn granite block pavement, would be well-nigh intolerable to the occupants of the buildings, as well as to those upon the streets.

The wooden pavements of Paris are similar to those of London in appearance, and the construction seemed to be practically the same. They are swept with brooms throughout the day and thoroughly flooded and washed every night. During the day the accumulations are swept to the curb and washed into the sewer by water turned on from hydrants opening flush with the sidewalk, near the curb.

The blocks of the stone pavements of both London and Paris are larger than those usually laid in this country. They are generally well worn at the edges and correspondingly rough and noisy. Asphalt sidewalks of ample width are universal in Paris. They are kept scrupulously clean.

The curbstones in Paris and throughout Italy are universally very substantial, being about 8 inches thick, and joined together with a tongue and groove. The curb is flush with the sidewalk, a little rounded or battered, and 6 or 7 inches above the paving material. There are no crossing walks, but in Paris and London there are, in the middle of the street, many places of refuge that serve a useful purpose.

In Rome and Naples there is a great deal of stone paving composed of stone blocks, about 4 inches square, and 6 inches deep, set upon a foundation (apparently) of broken stone or sand. The blocks are fitted closely, with very small joints which are without filling. This pavement is quiet, for stone, and seems to stand the traffic very well. The traffic, however, is not heavy.

There are many very handsome pavements in Pisa and Florence, composed simply of blocks of marble, or limestone, about 18 x 24 inches superficial measurement, by 7 to 8 inches thick, carefully dressed, fitted together with joints not to exceed ¼ inch, and laid upon a broken stone and sand foundation. The roadway is symmetrical, with a crown of about 6 inches in a 40-foot roadway. The turns at the street corners are kept rough by stone cutters with the ordinary hammer and point. At other places, the smoothness of the stones does not seem to affect the footing of the horses. No one, of whom inquiry was made, could tell how long the pavements had been down, and no one could remember when any of the streets had been paved. Repairing was in progress, however, and some new blocks were being placed. In the middle ages Florence was an important seat of learning, art and wealth, and it is likely

that her streets were well improved as early as the fourteenth century. Probably the same kind of pavements seen there to-day were then in use.

In mentioning pavements in the reverse order of their age and construction, a few words about the Appian way, one of the oldest and probably the most notable of all ancient highways (called the queen of roads) will not be out of place. It was built by Appius Claudius about the year 312 B. C., and connected Rome with Padua and a number of cities lying to the south and west.

The first sight of it is somewhat disappointing, unless one knows what to expect, for the reason that the roadway proper is only 14 feet wide ; there is, however, on each side, a sidewalk, 8 feet wide, which helps in some degree to restore the expected dignity.

The first two or three miles out of Rome are used as a modern highway, the original paving having been coverd with broken stone to make a smooth surface. The parts near Rome are not in ordinary use, but are kept open and preserved simply as a monument of antiquity. Only in a few places can the original paving be seen, most of it having been carried away by the dwellers in the Campagna and used for building stone fences and walls, and for other purposes. The curb stones, not being so easily removed, are in place throughout most of the way. The paving seemed to be of undressed limestone, or very hard and slippery basalt, laid flat, in irregular pieces of from 2 to 5 or 6 feet superficial area, and 6 or more inches thick. It is badly worn into ruts and very rough and uneven. The curb is of the same character of stone, and projects above the paving 6 or 8 inches. It had received some dressing with a hammer, but was not finely cut. The sidewalk was of the same general character of construction as the roadway.

The road is on high ground, and, for a distance of eight miles from Rome, and as much farther as the eye could follow it, probably six or seven miles, it is perfectly straight, with the exception of a slight detour, doubtless to avoid the three mounts supposed to have been built over the remains of the two Horatii, and three Curatii, who fell in combat before the assembled armies of Rome and Alba.

It was a custom with the Romans in those days, to inter persons of importance along the public highways. The Appian way, for a distance of eight miles, and perhaps much further, was lined on each side with tombs, some of which were doubtless of great magnificence. They have been sadly despoiled of all beauty, however, and, with the exception of the tomb of Cecelia Metella, which remains intact, have been entirely stripped of all decorations, and their marble covering. Nothing now remains but the brick and concrete core.

The paving of the Via Sacra, or Sacred way, in the Roman Forum, is similar to that just described. That in the streets of Pompeii is composed of large stones, but on the whole quite similar. It is very rough

and uneven, and much worn into ruts. Holes were pierced in the curb to serve as hitching places.

RIVER WALLS.

The substantial manner in which the rivers are confined, in their passage through the cities, would at once attract the attention of an engineer. This is particularly noticeable in Paris, Pisa, Florence and Rome, where the improvements of this character add greatly to the beauty of the districts near the river, transforming the space which is usually a mud flat, into a highway supported by massive walls, with a handsome coping, which serves as a rest for gaslights, statuary and other ornaments.

The walls at Paris, Pisa and Florence, do not exceed 30 feet in height above the ordinary winter stage of the water, but at Rome they are quite 45 feet, and built in the most substantial manner, the face stone being marble, carefully cut and laid in range courses, of considerable thickness, none being less than 15 inches.

The wall is built with a slight batter; and a foot-way, about four feet wide, is provided a few feet above the ordinary stage of the water. Stairways lead down at intervals, usually at the street crossings. Admittance could not be gained to the work which was in progress; but, from what could be seen, it seemed probable that the backing was composed of rubble masonry and concrete. The walls are built mostly in symmetrical curves, producing an effect very pleasing to the eye. This brings to my mind the argument often used against the levee system, as practised by the Government engineers to confine the water of the Mississippi, viz., that the bottom of the Tiber is now much higher than the valley adjacent, and as much as thirty feet higher than it was at the beginning of the Christian era, and that this condition was brought about by the levee system. This is certainly a mistake, for there are no visible evidences that the river is not cleaning its own bed; or that it was ever much lower than it is now. This opinion is strengthened by the fact that four or five feet of the water-way of the Cloaca Maxima was, in February of this year, visible above the surface of the water of the Tiber. This drain was built about the year 580 B. C., to drain the Roman Forum and adjacent territory, and it is said to have been the first application of the arch in Roman construction. It still performs its functions in a satisfactory manner, after having stood the vicissitudes of nearly twenty-five hundred years.

WATER SUPPLY.

To an engineer there is no more interesting study, in and about Rome, than the ancient water supply, and the remains of the aqueducts, which are everywhere to be seen in the southeastern part of the city, and in the country adjacent, for a distance of about seven miles in the

direction of the Alban hills. This stupendous work is worthy of the admiration of all beholders, and especially of the members of the engineering profession.

When the city had reached the zenith of its power and glory, no less than sixteen aqueducts, from six to sixty-one miles in length, and three branches, from two to three miles in length, had been built, and it then had a water supply estimated at 350 gallons per capita per day. It is better supplied with water to-day than any other city in the world, receiving daily, for each person, about three hundred gallons of clear, pure and palatable water.

The following table gives, in the chronological order of their construction, the total length of each aqueduct, the length of aqueduct supported on arches, and the date of construction, and conveys a faint idea of the magnitude of the work:

Name.	Date of Construction.	Total Length. Miles.	Length on Arches.	Remarks.
Appia	312 B. C.	11	Very little.	
Anio Vetus	272–264 B. C.	43	Very little.	
Marcia	145 B. C.	61	12 miles.	
Herculea Branch		3		
Tepula	126 B. C.	13	Very little.	
Julia	34 B. C.	15	6 miles.	
Virgo	21 B. C.	14	Very little.	
Alsientina	10 A. D.	22	Very little.	
Augusta	10 A. D.	6	Very little.	
Claudia	50 A. D.	46	10 miles.	
Anio Novus	52 A. D.	58	9 miles.	
Neronian Branch	97 A. D.	2	2 miles.	
Traiana	109 A. D.	42	Very little.	
Hadriana	117 A. D.	15	7 miles.	Restored 1585–1590.
Sabina Augusta	130 A. D.	15	Very little.	
Aurelia	162 A. D.	16	Probably 7 miles.	Elevated reservoir.
Severiana	200 A. D.	10	Not known.	
Antoniniana Branch	215 A. D.	3	3 miles.	
Alexandrina	226 A. D.	15	7 miles.	On Arches of Hadrian.
Totals		410	63 miles.	

FIG. 1.—MODERN ROMAN PAVING AND CURB AT PONTE ST. ANGELO.

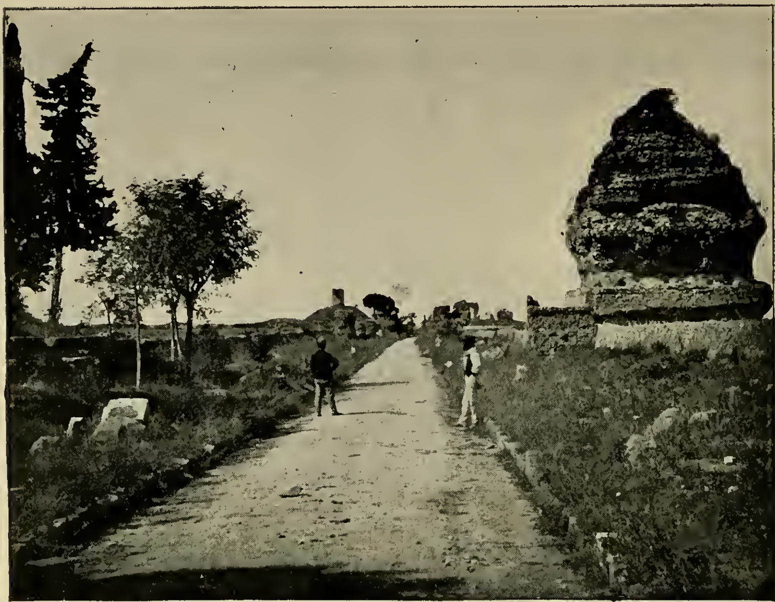


FIG. 2.—THE APPIAN WAY NEAR MOUNDS OF THE HORATII AND CURATII. ORIGINAL CURB VISIBLE; PAVING COVERED WITH BROKEN STONE.

FIG. 3—THE VIA SACRA IN THE ROMAN FORUM, SHOWING PAVING SIMILAR TO THAT OF THE APPIAN WAY.

[JOURNAL OF THE ASSOCIATION OF ENGINEERING SOCIETIES, Vol. XVII, No. 6, December, 1896.]

FIG. 4.—CONFINEMENT OF THE ARNO AT PISA.

FIG. 5.—CLOACA MAXIMA IN ROMAN FORUM.

FIG. 6.—VIEW OF THE CAMPAGNA NEAR ROME, SHOWING CLAUDIAN AND OTHER AQUEDUCTS AND THE NEW APPIAN WAY.

FIG. 7.—AQUEDUCT OF CLAUDIA.

The city, although upon seven hills, lies in the valley some seven miles from the foot of a range of hills, and, to reach its higher parts, it was necessary to begin to elevate the conduit by a series of arches at the foot of the hills, and continue them to the reservoir within the city; some parts were double and as much as 109 feet in height. The greater part of each, however, was less than 50 feet high, and composed of a series of single arches. In some cases, there were two conduits upon the same arch, one above the other. The supply from the Marcian Springs, by the Aqua Marcia, was one of the largest, and the favorite with the people because of its coolness and salubrity. It was brought in by seven conduits, supported by two rows of arches, parallel to each other, and about fifty feet apart. A part of this magnificent work still remains, but most of it was destroyed by Pope Sextus V, and the material used in the restoration of the aqueduct of Hadrian in 1585–90.

Some of the aqueducts were entirely under ground, and the major part of all of them were. Only in two or three instances does any part appear above ground until within seven miles of the city. All of those whose source is twenty or more miles distant, make long detours to avoid streams and valleys. For instance, the Marcian Springs are but thirty-six miles from Rome, yet sixty-one miles of aqueduct were built to conduct them thither.

Excepting the Anio Vetus and the Anio Novus, all the sources of supply were from springs or lakes, and the water clear and pure. The two named were taken from the river Anio, and, although settling and filtering basins were used, the water was at times turbid and unsatisfactory. In connection with this system were three large impounding reservoirs, created by dams built across the cañon of the Anio in the Symbraine mountains. The Anio Novus was an abundant supply, and, being the highest, it was sometimes used to reinforce the others; much against the will of the people, for at times it discolored any pure supply with which it was mixed. Most of the aqueducts were interchangeable at some point, and water could be turned out of one or more for repairs, without affecting the supply. The honor of beginning this system of water supply belongs to the censor Appius Claudius, better known as having built the road which bears his name. He constructed, B C. 312, the first aqueduct, which is almost entirely under ground, and which conveyed the waters from two springs to Rome, a distance of about eleven miles. Several of the older aqueducts were destroyed, and much of the material, used in their construction, was used again in later work of a similar character. Many of the most ancient were destroyed by the wars which finally caused the destruction of the Roman Empire, but the supply did not entirely cease until the fourteenth century.

In 1585–90 Pope Sextus V restored the aqueduct of Hadrian, and used therein, as stated, much of the material of the ancient Marcian and Claudian aqueducts. It is on arches for about six miles, and enters the city at the Porta Maggiore. It supplies many fountains of the present day.

Four of the ancient sources now supply the modern city, most of the water being brought in by iron pipes, only the upper or distant parts of the ancient conduit being utilized.

The conduits are from four to six feet high and always covered. When underground, they were lined with brick or stone. When the nature of the material required it they were plastered with cement which became very hard. At about every mile a bend was introduced, in order to break the force of the water and decrease its velocity, and at about every 240 feet, holes were left to admit air and to relieve the pressure should the conduits become too full. The Roman engineers had probably learned, by experience, that water would not flow freely in a closed conduit, not under pressure, unless air were introduced at frequent intervals, and therefore made openings near together for that purpose. It is by means of these respirators that the underground channels can be so readily traced.

It is to be inferred that the fall was ample, and greater than necessary, else they would not have introduced bends to check the velocity of the water.

The arches now standing are of stone or of brick, mostly of stone.

The generally prevalent idea that the Romans did not know that water would rise to the level of its source, is without doubt erroneous. Lead pipes, 3 inches in diameter, which had done service as water pipes in the palace of the Cæsars, are shown to visitors, and there is ample evidence that there was a distribution system of small pipes all over the ancient city. That they fully understood the natural laws which govern the flow of water is also unquestioned. They followed the contour of the country with their aqueducts, and doubtless built them, from controlling point to controlling point, upon a uniform grade, making tunnels when necessary, but, in the main, keeping the water line a short distance beneath the surface. This was the cheapest and most practicable method of construction with the materials available. When it became absolutely necessary to cross a valley, they raised the conduit above the surface of the ground upon arches of masonry, not because they did not know that water would rise again, but because they did not have, in sufficient abundance, material to construct a conduit that would carry a large quantity of water under pressure.

The longest tunnel in the vicinity of Rome was finished July 3, A. D. 88, and is under Monte Affliano, between Tivoli and S. Gericomio ;

it was 7 feet high, 3 feet wide and nearly three miles long. To supply the workmen with air is mentioned as one of the greatest difficulties encountered, but how this was accomplished is not stated. Lanciani records the discovery, in 1866, of a report upon some hydraulic work carried on in one of the African provinces, which throws some light upon the methods practiced by the engineers of those times. This report was engraved upon a marble altar, under the date A.D. 152, and reads as follows :

"Varius Clemens greets Valerius Etruscus, and begs him, in his own name, and in the name of the township of Saldæ, to dispatch at once the hydraulic engineer of the III legion, Nonius Datus, with orders that he finish the work, which he seems to have forgotten."

* * * *

Nonius Datus reported to the magistrates of Saldæ as follows:

"After leaving my quarters, I met with brigands on my way, who robbed me even of my clothes, and wounded me severely. I succeeded, after the encounter, in reaching Saldæ, where I was met by the governor, who, after allowing me some rest, took me to the tunnel. There I found every one sad and despondent. They had given up all hopes that the opposite sections of the tunnel would meet, because each section had already been excavated beyond the middle of the mountain, and the junction had not yet been effected. As always happens in this case, the fault was attributed to the engineer, as though he had not taken all precautions to insure the success of the work. What could I have done better? I began by surveying and taking the levels of the mountain; I marked most carefully the axis of the tunnel across the ridge; I drew plans and sections of the whole work, which plans I handed over to Petronius Celer, then governor of Mauritania, and in order to take extra precautions, I summoned the contractor and his workmen, and began the excavation in their presence, with the help of two gangs of experienced veterans.

* * * *

"What more could I have done? Well, during the four years I was absent at Lambæse, expecting every day to hear the good tidings of the arrival of the waters at Saldæ, the contractor and his assistant had committed blunder upon blunder. In each section of the tunnel they had diverged from the straight line, each toward his right, and, had I waited a little longer before coming, Saldæ would have possessed two tunnels instead of one."

The inscription further states that the connection was finally made by a transverse channel, and the arrival of the water celebrated by extraordinary rejoicings in the presence of the governor and the engineer.

To the mild, dry climate of Italy, more than to anything else, these

works owe their present state of preservation. In a more humid or colder climate, they would doubtless have been reduced to an unrecognizable mass. A comparison of the English with the Italian climate, in its effects upon the works of man, is aptly drawn by Hawthorne in the following lines from the "Marble Faun":

"The Italian climate, moreover, robs age of its reverence, and makes it look younger than it is. Not the Coliseum, nor the tombs of the Appian Way, nor the oldest pillar in the Forum, nor any other Roman ruin, be it as dilapidated as it may, ever gives the impression of venerable antiquity which we gather, along with the ivy, from the gray walls of an English abbey or castle, and yet every brick or stone, which we pick up among the former, had fallen, ages before the foundation of the latter was begun. This is owing to the kindliness with which nature takes an English ruin to her heart, covering it with ivy as tenderly as Robin Redbreast covered the dead babes with forest leaves. She strives to make it a part of herself, gradually obliterating the handiwork of man, and supplanting it with her own mosses and trailing verdure, till she has won the whole structure back. But, in Italy, whenever man has once hewn a stone, nature forthwith relinquishes her right to it, and never lays her finger on it again. Age after age finds it bare and naked, in the barren sunshine, and leaves it so."

Association of Engineering Societies.

Vol. XVII. JULY, 1896. No. 1.

PROCEEDINGS.

Boston Society of Civil Engineers.

June 17, 1896.—A regular meeting of the Society was held in Chipman Hall, Tremont Temple, Boston, at 8.15 o'clock, P.M. President George F. Swain in the chair. Sixty-five members and visitors present.

The record of the last meeting was read and approved.

Messrs. Thomas T. Allard, Wilfred A. Clapp, Rudolph Hering and Louis B. Vaughan were elected members of the Society.

The President announced the death of James H. Stanwood, a member of the Society, which occurred May 24, 1896. On motion, the President was requested to appoint a committee to prepare a memoir and the following were named as members of that committee: Arthur G. Robbins and Henry P. Bryant.

On motion, the sum of $60 was appropriated for binding.

On motion of Mr. Whitney, the thanks of the Society were voted to the Metropolitan Water Board for courtesies shown this afternoon on the occasion of the visit to Dam 5, at Southborough.

The literary exercises of the evening consisted of a very interesting paper by President Swain on the "History of Stone Bridges." The paper was profusely illustrated by lantern slides and traced the development of stone arch bridges from the earliest times to the present day.

Adjourned.

S. E. Tinkham, *Secretary.*

Association of Engineers of Virginia.—Summer Meeting.

The Association of Engineers of Virginia held its Summer Meeting at Maple Shade Inn, Pulaski, Va., June 26, 27, 1896, and the meeting was pronounced by those present the most successful and enjoyable in the history of the Association. This verdict was chiefly due to the generosity of the management of the Norfolk & Western Railroad, who on the 27th placed a special train at our disposal to visit the zinc, lead, and iron mines and reduction works in the celebrated Cripple Creek re-

gion. The enjoyment was enhanced by the presence of several ladies who accompanied their husbands or relatives to the meeting, and the success of this innovation will, we hope, make the presence of ladies one of the features of subsequent Summer Meetings.

The papers read at the meeting were timely and were listened to with great interest. The one by Mr. G. R. Henderson, on "Locomotive Counterbalancing," caused the asking of many questions, and much interest was shown in the new steel piston head recently introduced on the Norfolk & Western, and in other devices introduced with a view to lessening the weight of the reciprocating parts. The paper by Mr. W. H. Adams, of Mineral City, on "The Gold Belt of Virginia," was on a subject about which most of us knew but little, and we felt greatly indebted to the author for enlightenment. When it came to looking at the nuggets and ore specimens we all caught the gold fever and were thankful that we were all gold-bugs. The unanimous opinion seemed to be that the paper was a valuable one and should be published, but the author thought the time inopportune and made the condition before reading it that no part of it should be published at present.

The paper on "Good Roads," by Mr. J. R Schick, caused a discussion of the recent movement for good roads, and of the bill which was before the last legislature to enact a road law for the State. The bill was spoken of approvingly, and it was understood that we would help to secure its enactment by the next legislature.

The excursion on the next day was arranged in advance by Messrs. Coe and Churchill, and was carried out under the personal supervision of Mr. J. G. Osborne, Division Superintendent of the Norfolk & Western Railroad.

Captain J. C. Raper, Agent of Wythe Lead and Zinc Mine Co., was with the party during the whole day, and having been engaged in mining in that region for nearly forty years, could speak with authority on all matters pertaining to the region. At his works we saw the washing, jigging, and reduction of zinc and lead. Mr. G. M. Holstein, Vice-President and Manager of the Bertha Zinc and Mineral Co., was with us a part of the time, and placed every facility at our disposal, and Mr. McKee, Superintendent of Mines at Bertha, personally conducted the party at that place, showing the methods of working the zinc mines.

At Ivanhoe, Mr. George M. Seeley, General Manager New River Mineral Co., personally conducted the excursion through the iron mines, furnace, etc., belonging to his Company. At each place visited the local management did everything possible for our profitable entertainment, and when added to that we had the beautiful scenery of the New River valley, the presence of the ladies, and finally the excellent lunch furnished us by the Inn, we had a day long to be remembered by those fortunate enough to be able to attend the Summer Meeting of 1896.

ABSTRACT FROM THE MINUTES.

Maple Shade Inn, Pulaski, Va., June 26, 1896.—The meeting was called to order at 9 p.m. by the President, Prof. D. C. Humphreys. Fourteen members were present and a number of invited guests.

The following were elected members:

Robert E. Hutton, Lexington, Va.
F. H. Anschutz, Staunton, Va.
William Sleeper Aldrich, Morgantown, W. Va.
Theodore Low, Lynchburg, Va.
Ritchie Graham Kenly, Radford, Va.
M. J. Caples, of Bluefield, W. Va., was reinstated.

Mr. J. A. Pilcher, Secretary of the Association, stated in regard to the letter ballot sent out in order to ascertain the opinion of members on the bills pending in Congress for the establishing of the metric system, for the establishment of engineering experiment stations, and for the increase in the efficiency and personnel of the Navy; that while, with two or three exceptions to each bill, the replies favored the passage of the bill, they came in too late to be used at the recent session of Congress, but the result could be used in case the bills come up before another session of Congress.

A paper was read by Mr. G. R. Henderson on "Locomotive Counterbalancing," which was discussed and referred to the Committee on Publication.

Mr. W. H. Adams read a paper on "The Gold Belt of Virginia," which brought out many questions which the author answered in a most satisfactory way. At the request of the author the paper was not referred to the Committee on Publication.

Mr. M. E. Yeatman read a paper by Mr. James R. Schick on "Roads," which was discussed and referred to the Publication Committee. Adjourned.

On board excursion train June 27, 1896, before arriving at Pulaski on the return trip, a call meeting was held, presided over by the President.

The Association, by unanimous vote, thanked the Norfolk & Western R. R., The Bertha Zinc and Mineral Co., The Wythe Lead and Zinc Mine Co., The New River Mineral Co., and Mr. W. H. Hayes, manager of Maple Shade Inn, for courtesies extended to the Association.

Mr. J. G. Osborne, Division Superintendent Norfolk & Western R. R., was given a vote of thanks and reinstated to membership in the Association.

A vote of thanks was then given to the ladies of the party, and the meeting adjourned.

J. A. PILCHER, *Secretary.*

The Civil Engineers' Club of Cleveland.

THE July meeting of the Civil Engineers' Club of Cleveland, O., Tuesday evening, July 14, 1896, at the rooms of the School Council, President Howe in the chair. Present 82 members and visitors.

The minutes of the June meeting were read and approved. The Executive Board reported the resignation of Mr. C. W. Foote, and the applications of Messrs. John McGeorge and Carl C. Thomas.

Messrs. Aug. A. Honsberg and C. O. Palmer were appointed tellers to canvass the ballots for the election of Mr. Virgil E. Marani. Upon motion the question of the participation of the Club in the coming Centennial Celebration was referred to the Executive Board. It was voted to have a picnic, and also to have no August meeting. The paper of the evening, by Mr. H. F. J. Porter, of Chicago, was listened to with great interest. It was beautifully illustrated by lantern slides of photos and drawings. It gave an exhaustive description of the Bethlehem Iron Works at South Bethlehem, Pa., and their processes in the production of large forgings. The exhibition of photographic plates concluded with that of Mr. John Fritz, the founder of this great enterprise.

The speaker was followed by Messrs. Oldham and Newman, Dr. Langley, and others in interesting remarks, and Mr. J. F. Holloway appropriately finished the topic with a glowing tribute to the worth and ability of Mr. Fritz.

On motion of Mr. Mordecai, seconded by Mr. Cowles, an enthusiastic vote of thanks was tendered to Mr. Porter for his presentation of this paper.

President Howe announced the election of Mr. Marani, and named the Picnic Committee as follows: James Ritchie, W. O. Brown, A. L. Hyde, Hosea Paul and C. L. Saunders.

After the meeting a light lunch was served.

F. A. Coburn, *Secretary.*

ASSOCIATION OF ENGINEERING SOCIETIES.

Vol. XVII. **AUGUST, 1896.** No. 2.

PROCEEDINGS.

The Technical Society of the Pacific Coast.

Regular Meeting, August 7th, 1896.—Called to order at 8.30 p.m., by Vice-President Curtis. The minutes of the last regular meeting were read and approved.

Mr. Luther Wagoner read a paper entitled: "The Law of Equal Settling Particles," which was discussed.

Otto von Geldern, *Secretary*, per F. A. V.

Bradley & Poates, Engr's, N.Y.

Association of Engineering Societies.

Vol. XVII. SEPTEMBER, 1896. No. 3.

PROCEEDINGS.

Technical Society of the Pacific Coast.

Regular Meeting, Friday, September 4, 1896, held, according to previous arrangements, at the San Francisco Gas Works, at North Beach, through the courtesy of Mr. Edward C. Jones, who had invited the members and their ladies to an informal meeting of the Society.

President Dickie called the meeting to order.

Mr. Jones entertained and instructed the guests by reading a paper entitled "The Story of the X-Ray," and gave an interesting exhibition of an apparatus for creating the Roentgen Ray. A shadowgraph was taken in connection with the subject of the broken arm of a young lad, which was developed for inspection.

In addition to the reading of this paper, the members were conducted to the laboratory of the Gas Works, and were there shown the process of forming acetylene gas from calcium carbide, and the manner of using it for illuminating purposes.

An inspection of the gas plant closed the instructive entertainment of the evening, after which the meeting adjourned.

Otto von Geldern, *Secretary.*

Civil Engineers' Club of Cleveland.

Meeting of the Civil Engineers' Club of Cleveland, Tuesday evening, September 8, 1896, at the rooms of the Club, Case Library Building. Present, 33 members and visitors. Vice-President Ritchie in the chair. Minutes of the July meeting read and approved.

Messrs. A. Lincoln Hyde and Charles F. Lewis were appointed tellers to canvass the ballots for admission of John McGeorge and Carl E. Thomas to active membership.

Upon motion, committees were appointed by the chairman, as follows: Upon the death of Mr. Clarence O. Arey—Messrs. Coburn, Richardson and Hopkinson; upon the death of Mr. J. F. Holloway—Messrs. Swasey, Mordecai, Gobeille, Wellman, Strong and Paul.

Mr. Ritchie reported for the Picnic Committee, that the outing was a complete

success, the members having had a thoroughly enjoyable time; and that the committee had on hand twenty-five cents, after paying all expenses.

Mr. Swasey was then called to the chair, and the talk of the evening, upon "Some Examples in Recent Roof Construction," was given by Mr. Ritchie. The subject was thoroughly discussed by Messrs. Porter, Searles, Richardson, Hyde, Brown and others.

Messrs. McGeorge and Thomas were announced unanimously elected. The meeting then adjourned and the members proceeded to the restaurant.

F. A. COBURN, *Secretary.*

Engineers' Club of St. Louis.

439TH MEETING, SEPTEMBER 16, 1896.—The Club met at 8.35 P.M., at 1600 Lucas Place. President Ockerson in the chair and fourteen members present. The minutes of the 438th meeting were read and approved.

The Executive Committee reported the doings of its 217th, 218th, 219th, and 220th meetings. The Committee reported the establishment of a trust fund for the entertainment of visiting engineers, and referred to the Club the question of the best method of administering the trust. On motion, ordered that the fund be left in the hands of the Executive Committee, they to formulate a set of rules covering the matter, and to submit them to the Club for approval.

The Secretary was, on motion, directed to transmit to the local members of the American Society of Mechanical Engineers the thanks of the Engineers' Club of St. Louis for placing this trust in our hands, and for the contribution of the library fund of the Club.

The paper of W. J. Sherman, on the Galveston Harbor Works, was then read by Mr. B. L. Crosby, the author being absent. The paper gave a description of what is one of the most extensive improvements ever undertaken by the United States Government. The entrance to the harbor was impeded by a depth of only 12 feet on the outer bar, and 13 feet on the inner. The first improvements attempted were by dredging, without favorable results. The gabionade system was then undertaken, at great expense. No improvement resulted. The third project consisted of jetties of brush and stone. This bid fair to succeed, when it was found that the brush work was being destroyed by the sea worm known as the Teredo Navalis. The present scheme of two practically parallel solid rock jetties was then adopted, and is now in progress of construction. It has already deepened the outer bar to 20 feet, and it is believed that in good time it will reach the desired depth of 30 feet, with the aid of dredging.

The author gave the cost of the work, the rate of progress, and other interesting details.

Messrs. Crosby, Moore, Russell, and Barth participated in the discussion. Adjourned.

WILLIAM H. BRYAN, *Secretary.*

Boston Society of Civil Engineers.

SEPTEMBER 16, 1896.—A regular meeting of the Society was held in Chipman Hall, Tremont Temple, Boston, at 7.55 P.M. President George F. Swain in the chair; forty-two members and visitors present.

The record of the last meeting was read and approved.

Messrs. Frank H. Morris, Frank A. Peirce and Joseph H. Kimball were elected members of the Society.

The Secretary read a communication from the chairman of the Committee on Weights and Measures in relation to obtaining an expression of opinion of the members of the Society for or against the bill before Congress concerning the metric system. The communication stated that if the Society is desirous of obtaining such an expression, it will be necessary to make an appropriation of about $15 for printing and postage for the same. After a short discussion by Messrs. Brooks and Howland, a motion to appropriate the money asked for not having been seconded, the matter was allowed to drop without action.

Mr. George W. Blodgett then read the paper of the evening, entitled "Recent Practice in Railroad Signalling."

The paper was discussed by Messrs. Allen, Turner and Sampson.

On motion of Mr. Dorr the thanks of the Society were voted to Mr. C. D. Ingersoll, Jr., Resident Engineer, N. Y., N. H. & H. R. R., for courtesies shown this afternoon to members attending the excursion to inspect the work of raising the tracks of the Providence Division of that road.

Adjourned. S. E. TINKHAM, *Secretary.*

Bradley & Poates, Engr's, N.Y.

Association of Engineering Societies.

Vol. XVII. OCTOBER, 1896. No. 4.

PROCEEDINGS.

Technical Society of the Pacific Coast.

Regular Meeting, October 2, 1896.—This meeting was held in the great mercantile establishment, "The Emporium," on Market Street, in San Francisco, by invitation extended through the courtesy of the Management and the President, Mr. A. Feist.

The meeting was called to order by Mr. Dickie, who spoke of the engines and the special methods of condensation adopted.

Mr. Riffley explained the electric-lighting plant and installation, and other speakers followed in describing the different mechanical features of the establishment.

Mr. Feist addressed the Society on behalf of the Management.

An inspection was then made of the building, the pneumatic cashiering service, the elevators, the engines, the dynamos, the accountants' rooms and adding-machines, the restaurant, the kitchen, etc., etc., after which the meeting adjourned.

On motion of Mr. Grunsky, a vote of thanks was accorded Mr. Feist and the Emporium Company for the very interesting and instructive evening.

Attest, Otto von Geldern, *Secretary.*

Civil Engineers' Society of St. Paul.

St. Paul, Minn., October 5, 1896.—A regular meeting of the Civil Engineers' Society of St. Paul was held at 8.30 p.m.

Present, eleven members and one visitor. President Stevens in the chair.

The Secretary was instructed to accept, with thanks, the invitation of the echnical Club of Chicago to use club rooms.

A discussion of continuous rails on concrete foundation, in connection with asphalt pavements, was opened by Mr. Wilson and continued by Mr. Curtin, both well prepared to present results of practice and observation.

Adjourned at 10.30 p.m. C. L. Annan, *Secretary.*

Engineers' Club of St. Louis.

440TH MEETING, October 7, 1896.—The Club was called to order at 1600 Lucas Place, at 8.25 P.M., by Vice-President Flad. Seventeen members and one visitor present.

The minutes of the 439th meeting were read and approved. The executive committee reported the doings of its 221st meeting and submitted the following:

Rules and regulations governing the care, maintenance and disbursement of the engineers' entertainment fund:

The nucleus of this fund was derived from the local committee of the American Society of Mechanical Engineers, and a contribution from the Engineers' Club of St. Louis.

1. The fund shall be known as the Engineers' Entertainment Fund.

2. The Engineers' Entertainment Fund shall be devoted solely to the entertainment of distinguished engineers visiting our city, whether in conventions, small parties, or singly.

3. The fund shall be maintained as follows:

A. By interest on deposits.

B. By voluntary subscriptions.

C. By contributions from the treasury of the Engineers' Club of St. Louis, whenever in the judgment of the Executive Committee such contributions may be necessary and expedient—provided, however, that such contributions in any one year shall not exceed an amount equal to fifty cents for each resident member.

D. By special assessments, as provided in Section 2 of the By-laws.

4. The Executive Committee of the Engineers' Club of St. Louis shall have charge of the care, maintenance and disbursement of the Engineers' Entertainment Fund, subject to instructions from the Club when such action may be deemed necessary. Disbursements exeeeding $100 must first have the approval of the Club.

5. The affirmative vote of three members of the Executive Committee shall be required before any entertainment is undertaken. In emergencies, however, when a meeting of the Committee is impracticable—the president of the Club, or in his absence the vice-president, may authorize such entertainment.

6. Any member of the Club may recommend the entertainment of visiting engineers to the Executive Committee, accompanying such recommendation with sufficient evidence of the propriety of such action.

On motion the above rules were approved.

The Secretary read a letter from the Technical Club of Chicago, inviting the members of this Club to use their Club rooms when in Chicago during 1896. On motion, ordered that the invitation be entered on the Club's records, that it be accepted, and that the Secretary make due acknowledgment.

Applications for membership were announced from Sidney W. Fornham Mechanical Engineer Missouri Pacific Coal Companies, and Charles F. Womeldorf, Draughtsman Water Works Extension.

Mr. Alfred Siebert then read the paper of the evening, on "Refrigeration," as applied to dwellings, hotels, hospitals, business houses and public institutions. He explained the different methods of refrigeration which have heretofore been used, and the advantages and disadvantages of each, calling particular attention to the merits of modern refrigerating machines. The different uses to which such machines may be put are: the cooling of rooms, ice making, freezing of carafes,

making ice cream and cooling air in living rooms. The cooling may be done either by the direct or indirect system, each having its advantages under certain conditions. The cooling of rooms may readily be combined with the indirect heating system.

Discussions followed by Messrs. Kinealy, Barth, Johnson, Flad and Crosby.

Mr. Carl Barth gave the Club an interesting discussion of a geometrical method of determining the best points of cut-off and compression, which subject was also discussed by Messrs. Kinealy and Flad. Adjourned.

William H. Bryan, *Secretary.*

441st Meeting, October 21, 1896.—The Club met at 1600 Lucas Place, at 8.20 p.m., Vice-President Flad in the chair. Twenty-three members and four visitors present.

The minutes of the 440th meeting were read and approved. There being no other business, Mr. William H. Bryan then read a paper on "Boiler Efficiency, Capacity and Smokelessness with Low Grade Fuel." The discussion now going on among the mechanical engineers of this country regarding the best method of expressing the economic performance of boilers was explained, and the revision of the generally accepted code for making boiler trials shown to be necessary. The author strongly advocated the statement of boiler efficiency in the percentage realized of the calorific value of the fuel, taking care that the coal used be carefully sampled, and its calorific power determined by the most accurate means possible. The writer presented a table giving the results of a large number of trials he had made to determine the efficiency and smokelessness of various types of boilers, with and without improved settings. The table gave the maximum, minimum and average results secured. The paper was accompanied also by a table of fuel analyses and calorific determinations covering all the common Southern Illinois coals coming to this market.

Discussion followed, participated in by Messrs. Russell, Kinealy, Flad, Moore, Wheeler, Leighton, Ashburner, Harrington, and Wm. T. Bonner, of Cincinnati.

Adjourned. William H. Bryan, *Secretary.*

The Civil Engineers' Club of Cleveland.

Meeting held Tuesday evening, October 13, 1896, at the rooms of the Club in the Case Library Building; Vice-President Ritchie in the chair. Present, fifty-one members and visitors.

The minutes of the September meeting were read and approved.

The Secretary reported for the Executive Board the resignation of Mr. E. W. S. Moore, and the applications for active membership, of Messrs. Alex. Raynal and Wm. C. Thayer.

Committees presented resolutions upon the death of Mr. J. F. Holloway and of Dr. C. O. Arey, as below. They were unanimously adopted by a rising vote.

Remarks appropriate to the occasion were offered by Messrs. Warner, Gobeille, Kimball and Searles, and letters were read from Mrs. Holloway and Mr. F. H. Richards.

A letter from the Technical Club of Chicago, extending an invitation to members of this Club to visit their club-rooms whenever in Chicago, was read.

Mr. Varney then presented the paper of the evening, entitled "Solar Work in Land Surveying." Mr. Varney's clear description of the principles governing solar work, and of a new device for use in that method of land surveying, was very fine and well appreciated by the members of the Club. In the discussion which followed, Messrs. Culley and Baker took an interesting part.

After the meeting a light lunch was served.

F. A. Coburn, *Secretary.*

Cleveland, O., October, 1896.—A semi-monthly meeting of the Club was held on Tuesday evening, October 27, 1896, at the rooms in the Case Library Building. Present, fifty-seven members and visitors.

Mr. Cecil L. Saunders read a paper entitled "Gas Producers and the Mechanical Handling of Fuel." The subject was presented under the following heads: A Discussion of Various Types; The Necessity for Attention to Detail of Construction; The Relation of Character of Coal to Type to be Used; A Possible Field for Future Economy; Coal Handling from Hoppers; Unloading Coal by Mechanical Devices.

Messrs. Sperry, Mordecai, Barber and others took part in an interesting discussion.

After the meeting a light lunch was served.

F. A. Coburn, *Secretary.*

CLARENCE O. AREY, C.E., M.D.—A MEMOIR.

In Dr. Clarence O. Arey, the Civil Engineers' Club of Cleveland lost one of its brightest and most honored members.

We recall that he came to Cleveland a graduate of the School of Engineering of the State University of Michigan, having been engaged, immediately after leaving college, with prominent architects in Buffalo and Chicago.

We remember the enthusiasm with which he entered the architectural field of Cleveland, and his aim to combine the training of an engineer with the experience of an architect. For ten years he labored faithfully and well, winning laurels on every hand for his sober, consistent, scholarly work and his honorable record. Having more of the training and spirit of a civil engineer than was generally common with architects, he naturally sympathized strongly with our Civil Engineers' Club and its work, and was more than ordinarily interested in the broadening of the two professions in their mutual relations. His able and interesting papers, delivered before us from time to time, abundantly illustrated his spirit, his ambitions and his mental strength. And when, through personal losses in his family, his mind turned toward the great field of bacteriological work, and he elected to carry his experience, his studious habits, and his conscientious zeal into this new field of research, we felt that truly no one of our members was better fitted to enter it.

His success in his new work, which was to be the chosen work of his middle and later life, was soon manifested. We recall the confidence which he won from the professors and physicians connected with the Western Reserve Medical College, where his active bacteriological work was conducted. Surely he would have reaped large honors had he been spared to prosecute this work. Those who listened to the masterly paper on "Water Supply and Sewerage as affected by the Lower Vegetable

Organisms," which was read at the June meeting of this year, will remember the ability shown, and the patient study exhibited.

He was about thirty-nine years old at his death, an age when life is seen with the cool, dispassionate vision of fully matured manhood, when the powers take on new vigor and strength.

Honorable and upright in the smallest, as well as in the largest matters entrusted to his care, he was one such as the world delights to honor.

To his widow and his two little children we extend our heartfelt sympathy and commiserations.

Therefore be it Resolved, that we hereby express our deep-felt regrets at Dr. Arey's death and that we forward a copy of the same to his bereaved family, and also spread upon the minutes of the Club a copy, that future members may know of our sincere regard for him.

F. A. COBURN,
JNO. N. RICHARDSON,
CHAS. W. HOPKINSON, } *Committee on Resolutions.*

JOSEPHUS FLAVIUS HOLLOWAY.—A MEMOIR.

MR. JOSEPHUS FLAVIUS HOLLOWAY was born in Uniontown, Stark County, Ohio, January 18, 1825, and he died at Cuyahoga Falls, Ohio, September 1, 1896.

It is impossible for the Civil Engineers' Club of Cleveland to express its sense of loss, in words that will appear other than trite and commonplace, at the death of Josephus Flavius Halloway, for his death is not only a great loss to this Club, but much more so to its individual members. He was our friend and our adviser; he knew our names, our work, our circumstances, and was ever ready with congratulations and praise for our individual successes, as well as with encouragement and sympathy when we were cast down. It is exceedingly difficult to come together as a Club, and express in words, that which is more plainly told by the moistened eye and saddened brow, for our memory pictures before us his ever gentle and genial personality whenever we hear the mention of his name.

We are proud to speak of his achievements, for he devoted his life to the field of engineering at a time when there were almost no technical schools and very few books of reference; when it was necessary to solve great problems by personal experiment, such as now can be put upon the board in a few minutes by the use of simple formulæ; at a time, too, when progress (which necessarily involves change) was forced upon this country because of the necessities of utilizing our great lakes and rivers as means of transportation; of building our railways, opening our mines, and meeting the demands of urban life. It certainly was not easy then for this simple country boy to achieve a reputation which is wider than the continent. He was one of the founders of this Club, for three successive terms its President and one of its honorary members; he was chosen President of the American Society of Mechanical Engineers, Vice President of the American Institute of Mining Engineers, and President of the Engineers' Club of New York. He held the position of President and Engineer of the Cuyahoga Steam Furnace Company of Cleveland, Vice-President and Consulting Engineer of the Worthington Hydraulic Works, and after that, until the time of his death, Consulting Engineer for the Snow Steam Pump Works. But we need not proclaim his standing as an engineer, so we pass his eminent professional career to note other characteristics and accomplishments just as prominent.

First was his high standing as a Christian gentleman. No question of morals or ethics but found Mr. Holloway on the right side, and all his life long its ardent champion. He was possessed of intense desire to promote the universal brotherhood of man, and was especially full of sympathy for the artisan; as is indicated by his address to the workmen of the Cuyahoga Steam Furnace Company after the sale of that plant, one of the most pathetic bits of literature ever written in that line.

In his domestic life he was perhaps seen at his best, and to his sorrowing widow and to his son and daughter, the sympathy of those who had the good fortune to share his hospitality, is instinctively tendered. Happily his son bids fair to emulate his father's example and standing.

Mr. Holloway was especially noted for his literary accomplishments. His contributions to the engineering literature of the day were of the highest order, and always interesting and instructive. Very early in life he conceived a warm admiration for Dickens' works, and he accumulated later a noteworthy library of volumes bearing upon this author and his works. It is doubtful whether any man in the United States had a better knowledge than he of the works of Dickens, of the characters he created, or of the motives and sympathies which inspired the author in formulating the plots of his books.

Socially, Mr. Holloway was one of the most lovable and enjoyable of men. His quiet and even disposition, his ready and clear wit—never sharpened by pointed or ill-natured remarks—left nothing but pleasant memories behind. His thorough appreciation of others, and his consideration for them, made him a prince of entertainers; and those who remember him as speaker or as toast-master at our banquets, know how much we owe to this gentle, quiet, and sympathetic character. To the younger and more retiring members of the profession, he was especially encouraging and helpful, being ever a welcome counselor among them, and so it was with all with whom he was associated.

Indeed, the following words spoken by him in memory of his beloved friend, Alexander Lyman Holley, are equally true of Josephus Flavius Holloway: "When his biography shall be truly written, it will be found, that while his accomplished work and deeds as an engineer will give him a place among the ablest in the profession he so well adorned, his highest and best monument will be found in the loving memory of him, that will ever linger in the hearts of his friends."

AMBROSE SWASEY,
AUGUSTUS MORDECAI,
JOSEPH LEON GOBEILLE,
S. T. WELLMAN,
C. H. STRONG,
HOSEA PAUL,

Committee on Resolutions.

Association of Engineering Societies.

Vol. XVII. NOVEMBER, 1896. No. 5.

PROCEEDINGS.

Civil Engineers' Society of St. Paul.

A regular meeting of the Civil Engineers' Society of St. Paul was held on November 2, 1896, at 8.45 p.m.

Present ten members and six visitors. President Stevens in the chair.

The discussion of the evening was led by Mr. H. H. Vaughan, M. E., of the Great Northern Railway Co., and was suggested by incidents of a recent visit to the shops of the company by fourteen members of the Society.

These shops are equipped with laboratories for mechanical and chemical tests and pneumatic appliances are in general use in the various departments.

The discussion was continued and somewhat broadened over the tables at Neuman's after 10 o'clock. C. L. Annan, *Secretary.*

Engineers' Club of St. Louis.

442d Meeting, November 4, 1896.—President Ockerson called the Club to order at 8.30 p.m., at 1600 Lucas Place. Twenty-six members and four visitors present.

Mr. Julius Baier was, on motion, elected Secretary *pro tem.* The minutes of the 441st meeting were read and approved. The doings of the 222d meeting of the Executive Committee were reported, recommending the applications for membership of Messrs. S. W. Farnham and Chas. F. Womeldorf. They were balloted for and elected. The receipt of the following publications for the library was announced: Annual Report of the Massachusetts State Board of Health, Appletons' Cyclopedia of Drawing and Design, presented by Mr. W. E. Worthen, and Seventh Annual Report of the Syracuse, N. Y., Water Board. The Secretary was directed to make due acknowledgment. The Secretary read a letter from the American Street Railway Association acknowledging courtesies extended by this Club during their recent convention.

Mr. M. L. Holman then presented informally the proposed specifications and form of contract prepared by the Board of Public Improvements for the lighting of

the streets, alleys and public places of the city of St. Louis for a term of twenty years, beginning in 1900. The most important features of the proposed contract were the exclusive use of 32 candle-power incandescent lights in place of the arcs of 2000 nominal candle-power, all wires to be underground.

St. Louis was the first large city to adopt electric lighting on a large scale, and has therefore had a very wide and valuable experience, arcs being used for the streets generally, and incandescents for the alleys, parks, and also for a number of snburban streets. Valuable practical experience had therefore been had in the use of both kinds of lights, and the adoption of the incandescent light to the exclusion of the arc is the result of a careful investigation into the illumination given by the two systems.

The duration of the contract was made twenty years in order that the city might secure reasonable bids.

Diccussion followed by Messrs. R. E. McMath and B. H. Colby, who, with Mr. Holman, formed the sub-committee of the Board which had this work in hand. They emphasized the fact that their conclusion was based upon actual observations made on the two systems of lighting in regular service in this city. The arc lights give a very unequal distribution, the illuminations being very intense at one point and there being but little light midway between. The incandescent lights, on the other hand, are placed much nearer together and afford a much more uniform light.

Further informal and general discussion followed, in which Messrs. Robert Moore, Eayrs, Crosby, Barth, Van Ornum, Ockerson, Pitzman, Wise and Philip Moore participated. Adjourned.

Julius Baier, *Secretary pro tem.*

443d Meeting, November 18, 1896.—The Club was called to order at 8.40 p.m. at 1600 Lucas Place. Vice-President Flad in the chair, twenty-two members and four visitors present. The minutes of the 442d meeting were read and approved. The Executive Committee reported the doings of its 223d meeting. An application for membership was announced from W. N. Graves, general superintendent Hydraulic Press Brick Company, endorsed by F. H. Pond and William H. Bryan.

The Club then proceeded to ballot for a Committee on Nomination of Officers for 1897, the Committee to report at the next meeting. The balloting resulting in the selection of S. E. Freeman, F. B. Maltby, J. A. Laird, Julius Baier and S. B. Russell.

Mr. Carl Barth then gave the Club an informal talk on the Emery Testing Machine, in the development of which he took a prominent part. This machine, which was the invention of Mr. A. H. Emery, C. E., is one of the most wonderful inventions of the age, embodying many new principles, and working absolutely without friction. It is capable of giving the most accurate results, whether the load be large or small. Mr. Barth exhibited a number of lantern slides, showing the general appearance of the machine, and its most important details. Brief discussion followed, participated in by Messrs. Freeman, Harrington, Baier and Russell.

Prof. J. B. Johnson showed the Club a new form of cement briquette, which he had designed with a view of securing more accurate results in cement testing. He

showed wherein the ordinary form of briquette was imperfect, and gave the theoretical considerations governing his design and the results it has given in practice. Adjourned.

William H. Bryan, *Secretary.*

Technical Society of the Pacific Coast.

Regular Meeting, November 6, 1896.—Called to order by President Dickie. The minutes of the last regular meeting were read and approved.

The application for membership of Edward F. Haas, proposed by H. I. Randall, Frank Soulé and Hermann Kower, was referred to the Executive Committee for the usual action.

Professor Albert T. Smith spoke to the members on the subject of "Proper Shapes in Machine Design," illustrating his lecture by numerous diagrams on the blackboard. This interesting subject was discussed by President Dickie, A. d'Erlach, Mr. Buchanan and others.

Adjourned. Otto von Geldern, *Secretary.*

The Civil Engineers' Club of Cleveland.

The November Meeting of the Club was held in the rooms of the School Council, Public Library Building, Tuesday evening, November 10, 1896. Mr. W. H. Searles was chosen chairman. Present fifty-two members and visitors. The minutes of the two preceding meetings were read and approved. Messrs. S. J. Baker and August Honsburg were appointed tellers to canvass the ballots for admission to active membership of Alfred H. Raynal. Mr. Hyde reported, at the request of the Chairman, in regard to the progress of the proposed amalgamation of the technical societies of this city.

Joseph R. Oldham, N. A. and M. E., then read the paper of the evening, on "Structural Strength of Ships and Improved Arrangements for Repairing without Diminution of Strength."

Mr. Oldham's paper treated of matters under the following heads: Progression by Steps in Engineering; Increase in Steel Lake Tonnage; Bending Moment and Shearing Stress; Strength of Beams and Girders; Straining of Ships; Improved Hatches; Useful Weak Ships; Joggling and Lapping; Flush Bottoms; Heavy Ships; Light Ships; A Perfect Mechanical Structure.

Messrs. Newman, Searles and Head followed in an interesting discussion. Mr. Raynal was reported unanimously elected.

After the meeting a light lunch was served.

F. A. Coburn, *Secretary.*

Montana Society of Civil Engineers.

The November meeting of the Montana Society of Civil Engineers was held Saturday evening, November 14th.

The applications for membership of William A. Clark, of Butte, and Frank

Leonard, of Libby, were read and the Secretary was directed to send out letter ballots to the members, to be canvassed at the next regular meeting.

Prof. Ryon's report on Senate bill No. 2301, which provides for the establishment of engineering experimental stations in each State by the general govern ment, was approved by the society.

A letter from Prof. Ryon, in which he expressed a desire to withdraw from active membership owing to the fact that he had left the State, was read. He was placed upon the list of associate members.

Messrs. Carrol, Taylor and Bickel were appointed a committee to place in nomination a list of candidates for officers for the ensuing year. Messrs. Keerl, Blackford and Parker were named a committee to arrange for the annual meeting, to be held the second Saturday in January. The meeting will be held in Great Falls, and while there the members will visit the smelters, the dam, the giant spring and the Sand Coulee coal mines and other points of engineering interest.

F. W. Blackford, City Engineer of Butte, read a paper on "A Few Points of Interest Observed on a Short Trip Abroad. Pavements, Confined Rivers and Water Supply of Ancient Rome." His description of the Appian Way and the modern pavements of London and Paris left little doubt that American cities have still much to learn. Mr. Blackford also exhibited a fine collection of views of some of the various points of interest which he visited.

After the paper was discussed it was referred to the trustees, who will publish it in the JOURNAL OF THE ASSOCIATION OF ENGINEERING SOCIETIES.

There were present at the meeting: F. W. Blackford and Eugene Carroll, of Butte; Maurice Parker, of Great Falls; and John Herran, A. E. Cumming, F. J. Smith and A. S. Hovey, of Helena. The visiting engineers were J. M. McGregor, of Rossland, B. C.; Mr. King, of Great Falls, and F. L. Sizer, of Helena.

F. J. SMITH, *Secretary.*

Association of Engineering Societies.

Vol. XVII. December, 1896. No. 6.

PROCEEDINGS.

Boston Society of Civil Engineers.

October 21, 1896.—A regular meeting of the Society was held in Chipman Hall, Tremont Temple, Boston, at 7.50 p.m. President George F. Swain in the chair, 95 members and visitors present.

The record of the last meeting was read and approved.

Messrs. Rowland H. Barnes, Edward D. Bolton, Isaac W. Hastings, Alfred T. Palmer and Charles H. Peck were elected members of the Society.

The Secretary read a memoir of James H. Stanwood, a member of the Society, prepared by a committee of the Society, consisting of Messrs. A. G. Robbins and H. F. Bryant.

The President announced the death of Past President Albert F. Noyes, which occurred on October 12, 1896, and in accordance with the usual custom appointed as a committee to prepare a memorial, Messrs. G. A. Kimball and H. D. Woods.

Mr. Main, for the Committee on Weights and Measures, presented a report, giving the result of a canvass of the Society to obtain the consensus of opinion on the proposed action of Congress with relation to the use of the Metric System. Mr. Howland objected to such an announcement being made, for the reason that the Society at its last meeting had declined to appropriate money for the purpose of making this canvass. The President ruled that the Committee on Weights and Measures could submit such a report, and upon an appeal being taken from the ruling of the chair, his decision was sustained. Mr. Main then read the following report:

The Committee on Weights and Measures was requested to obtain a consensus of opinion on the proposed action of Congress with relation to the use of the Metric System.

At the last meeting of the Society the estimated cost of getting this consensus was stated to be about $15, but the Society did not see fit to make an appropriation for this purpose. Since the meeting, the necessary amount has been received by the Committee independent of the Society.

Postal cards reading as follows were sent to each member:

> " I am———in favor of the passage by the present Congress of an Act requiring the metric weights and measures to be in use by the government departments generally by the beginning of the Twentieth century, January 1, 1901.

"I should———be willing to have people generally of their own acord adopt metric weights and measures for their ordinary business transactions, and especially for those in which I am myself concerned, at the same time at which the government departments as a whole actually do adopt them."

The total number of cards sent out was	404
The total number of cards returned was	229
Number in favor of both clauses of the card	193
Number against both clauses of the card	21
Number for first clause and against second	2
Number against first clause and for second	11
Members in favor of a decimal system	2

Respectfully submitted,

CHAS. T. MAIN,
For the Committee on Weights and Measures.

Mr. John L. Howard was introduced and read a paper entitled "A brief Account of Topographical Work on Mr. George W. Vanderbilt's North Carolina Property."

Mr. Henry F. Bryant followed with a paper entitled "Topographical Surveys of the Metropolitan Park Reservations of Massachusetts."

The papers were fully illustrated by maps showing the work covered.

A general discussion followed, in which Messrs. A. H. French, W. E. McClintock, E. P. Adams and others took part.

Adjourned. S. E. TINKHAM, *Secretary.*

NOVEMBER 18, 1896.—A regular meeting of the Society was held in Chipman Hall, Tremont Temple, at 7.45 P.M. President George F. Swain in the chair. Total number present 227. The members of the Society of Arts of the Massachusetts Institute of Technology were invited to join in this meeting and a number availed themselves of the invitation.

The record of the last meeting was read and approved.

Messrs. Alton D. Adams, Fred Lavis and Eugene E. Pettee were elected members of the Society.

The Secretary presented, for the Board of Government, the following rules with regard to the circulation of the books of the Library, which the Board recommended to be adopted in place of those now in force:

Books and periodicals may be used in the Reading-Room by members and friends, and by students recommended by the Boston Public Library.

Members may borrow books for home use, but no one shall have more than four books at any time, nor keep any book more than five weeks.

A member borrowing a book shall give a receipt therefore to a member of the Library Committee, to the Secretary, or to the regular attendant.

A fine of one cent per day per volume shall be charged for overtime, and must be paid before the delinquent can take any more books.

Current numbers or unbound files of periodicals shall not be taken from the room.

Books of unusual value are marked with a star (*), and must not be taken from the room, except by written permission from the Board of Government.

Any person mutilating or losing a book shall pay for the damage, or replace the book.

Any one who violates the above rules shall, upon written request from the Librarian to the Board of Government, be debarred from the privileges of the library for such time, not less than three months, as the Board of Government may determine.

On motion of Mr. French the recommendation of the Board was adopted.

On motion of Mr. Thompson the thanks of the Society were voted to the George F. Blake Manufacturing Co., for courtesies shown the members this afternoon on the occasion of the visit to the Company's works at East Cambridge.

Mr. E. L. Corthell was then introduced and delivered a lecture entitled "The Tampico Harbor Works, Mexico, with Some Remarks upon the Mouth of the Mississippi River."

The lecture was profusely illustrated with lantern slides and plans.

Adjourned. S. E. Tinkham, *Secretary.*

December 16, 1896.—A regular meeting of the Society was held in Chipman Hall, Tremont Temple, Boston, at 7.50 p.m. Vice-President Dexter Brackett in the chair. Sixty-two members and visitors present.

The record of the last meeting was read and approved.

Messrs. Samuel D. Dodge, Frank E. Fuller, Ralph E. Marston, Elmer W. Ross, and Henry A. Symonds were elected members of the Society.

The Secretary read a memoir of Forrest L. Libbey, a member of the Society, prepared by a committee appointed for that purpose.

Mr. F. Herbert Snow then read the paper of the evening on Sewer Assessments.

The paper was discussed by Messrs. Hazen, G. A. Kimball, C. R. Cutter, F. P. Stearns, Coffin, Hawes and others.

The Secretary also read in full, discussions prepared by Messrs. T. H. Barnes and George Bowers, and by titles a number of other contributions, which would be printed with the proceedings.

President Swain assumed the chair and on motion of Mr. Whitney, the thanks of the Society were voted to the managment of the New York, New Haven & Hartford Railroad for its generosity in placing a special car at the disposal of the Society for the proposed excursion to Providence.

Adjourned.

S. E. Tinkham, *Secretary.*

James Hugh Stanwood.—A Memoir.

By A. G. Robbins and H. F. Bryant, Committee of the Boston Society of Civil Engineers.

[Read before the Society, October 21, 1896.]

James Hugh Stanwood was born at Brunswick, Me., July 17, 1860. He began his professional life in 1879, when he entered the employ of Edward C. Jordan, Civil Engineer, of Portland, Me.

During the two following years he was engaged in general engineering-work in and near the city of Portland.

From 1881 to 1883 he served as leveler and transmit-man in the engineering department of the Maine Central Railroad.

In order to more thoroughly equip himself for his profession, he entered the Massachusetts Institute of Technology in 1883, and four years later graduated from its civil engineering department.

In 1887 he entered the office of the designing engineer of the Philadelphia

Bridge Works, at Pottstown, Pa., where he remained a year, when he resigned his position in order to return to the Institute of Technology as an assistant in civil engineering. Two years later he became an instructor, and for several years previous to his death, he was in charge of the drawing-room work in bridge and roof design. During this time he was also an instructor in mechanical drawing in the Boston evening schools.

Among his pupils he was considerate, uniformly courteous, and untiring in his efforts to make clear the principles underlying the subject, and to point out the application of those principles to the particular design in hand.

Among his friends and acquaintances he was always frank and manly, with an abundance of good humor, which, together with his high character and generous disposition, made him most esteemed among those who knew him best.

Of his contributions to engineering literature, perhaps the most widely known is his "column formulas" for yellow pine posts, printed in the *Railroad Gazette* in 1892 and 1894.

He was a member of this Society from September 17, 1890, till his death on May 24, 1896.

He was also elected an associate member of the American Society of Civil Engineers on October 3, 1894.

Forrest L. Libby.—A Memoir.

By Henry Manley, S. E. Tinkham and N. S. Brock, Committee of the Boston Society of Civil Engineers.

[Read December 16, 1896.]

Forrest Llewellyn Libby was born at Great Falls, N. H., August 19, 1864. He came to Boston in 1869, and was educated in the Boston Public Schools, graduating from the Roxbury High School in 1879. On November 1, 1881, he entered the office of the City Engineer of Boston, and remained in its employ until his death on July 21, 1894. For the first eight years of this time he was employed in the central office, principally upon the construction of bridges, in connection with structures of an allied character, such as wharves, sea-walls, etc.

During this time he had a part in the construction of the following structures, besides much other miscellaneous work: Warren and Meridian Street Bridges, Albany Street, Broadway and Boylston Street Bridges, over the Boston & Albany Railroad, Wharves at Long and Deer Island, and East Boston Ferries.

In December, 1889, an opportunity for promotion transferred him to the force employed upon Dam No. 6, of the Boston Water Works, and upon this work and upon surveys and studies for the further extension of the water works system he was employed until his death.

It early became evident that he was suffering from the incurable malady which finally caused his death. He continued unflinchingly at his post, however, until December 24, 1892, when by advise of his physician and friends he went to Southern California, in the hope of restoring his now much impaired health. This hope proved vain, for his return in May 1893 found him little improved.

With the courage and perseverance which characterized him to the last, he again attempted active work in July, 1893, and, although frequently compelled to give up for intervals of a few days, he continued the struggle till January 25, 1894, when he broke down completely; from this time he failed steadily until his death on July 21st.

Although ill for many months and at times suffering severely, he bore his part of the work faithfully and willingly, never asking for favor and never giving up till absolutely compelled to.

He joined the Society November 15, 1885. He was unmarried.

He leaves behind him many friends, who remember his prompt, quick ways and active habits, who admire the courage always shown by him and particularly in his long fight with a wasting disease, and who sympathize with his family in the pathetic ending of a life cut short before its time.

Engineers' Club of Minneapolis.

MINNEAPOLIS, MINN.—A meeting of the Engineers' Club of Minneapolis was held November 16, 1896, at 8.00 P.M., in the Council Chamber, City Hall. President F. W. Cappelen in the chair.

The committee appointed at the last meeting to solicit subscriptions to extinguish the indebtedness of the Club, reported verbally that they had accomplished the object of their appointment, and submitted a draft on New York for $77.28 as their written report.

Their report was accepted, and, on motion, the thanks of the Club were extended to them and to the several subscribers to the same, and the Secretary was directed to notify them of the Club's action.

Mr. F. W. Cappelen then read a paper on "Cost of Electric Lighting in the United States."

After informal discussion, a motion was passed, "That a committee, to further discuss this subject, and to make further deductions from the voluminous table submitted by Mr. Cappelen with his paper, to be presented to the Club and published in the JOURNAL, be appointed." On motion, adjourned.

ELBERT NEXSEN, *Secretary.*

MINNEAPOLIS, MINN.—A meeting of the Engineers' Club of Minneapolis was held at the office of the City Engineer, City Hall, Tuesday, December 29, 1896, at 8 P.M. President F. W. Cappelen in the chair.

Minutes of previous meetings were read and approved, after correcting those of the last meeting relative to report of Committee on Debt Extinguishment, as follows: The Committee, having failed to accomplish anything, the President took the matter in hand with the success indicated in the report.

There were read communications from Charles B. Billin, Secretary, extending to our members an invitation to use the rooms of the Technical Club, of Chicago, when in that city; an invitation from the Western Society of Engineers to our President and Secretary to attend their annual banquet, and a letter from John C. Trautwine, Jr., Secretary Association of Engineering Societies, enclosing sample letter-head and offering electrotype of map for our use.

The Secretary was directed to thank the Technical Club of Chicago, and to accept Mr. Trautwine's offer.

The draft of a bill, embodying proposed legislation on the subject of licensing "Measurers of Land," presented by Mr. William Danforth, of Redwing, was read, and referred to a committee, consisting of E. T. Abbott, Ellis R. Dutton and J. E. Egan, appointed, under a motion, "That a committee of three be appointed to act with committees from Civil Engineers' Society of St. Paul, the Surveyors' Society of

Minnesota and the University of Minnesota, to prepare a bill and endeavor to have it passed, as will correct the many faults of the present laws relative to measuring, or surveying of land."

Charles E. Pillsbury and C. H. Kendall were unanimously elected members of the Club.

Mr. George D. Shepardson, C.E., read a paper on "Some Principles of Artificial Lighting," which was discussed, and he was requested to furnish a copy for publication. On motion, adjourned.

Elbert Nexsen, *Secretary.*

Engineers' Club of St. Louis.

444th Meeting, December 2, 1896.—The annual meeting was held at 1600 Lucas Place. Vice-President Flad called the meeting to order at 8.10 p.m. Thirty members and twelve visitors present, three of the latter being ladies. The minutes of the 443d meeting were read and approved. The Executive Committee reported the doings of its 224th meeting approving the application for membership of W. N. Graves, general superintendent Hydraulic Press Brick Company. He was balloted for and elected. An application for membership was announced from C. E. Delafield, engineer of construction St. Louis Electric Light and Power Company, endorsed by B. H. Colby and A. H. Zeller.

The Secretary then read his annual report giving a summary of the Club's work for the year just past. On motion, this report was ordered received and filed. Thos. B. McMath, treasurer, then read the annual report of the Club's finances. On motion, ordered referred to the Executive Committee to be audited. The Committee on Improvement of Library, Julius Baier, chairman, then made a report which was on motion accepted and the committee discharged with thanks, its work having been completed.

Prof. J. B. Johnson made a report for the Board of Managers stating that only routine business had been transacted during the past year.

Vice-President Flad then submitted a report summarizing the work of the Executive Committee for the year, showing a gratifying improvement of the Club's finances. On motion received and filed.

Col. E. D. Meier, Chairman of the Committees on Monument to Capt. Eads and Standard Gauges for Thickness, stated that these committees had nothing special to report, and asked that they be continued.

The Secretary stated that he was in receipt of a letter from the Librarian stating that continued absence from the city had prevented his preparing a formal report.

The Committee on Smoke Prevention made no report.

S. Bent Russell read the report of the Nominating Committee as follows:

For President: Edw. Flad.

For Vice-President: William H. Bryan.

For Secretary: Richard McCulloch.

For Treasurer: Thos. B. McMath.

For Librarian: W. A. Layman.

For Directors: J. A. Ockerson and B. H. Colby.

For Board of Managers: J. B. Johnson and E. A. Hermann.

Other nominations being called for, Mr. Carl Gaylor was nominated for vice-president.

The Secretary called attention to a letter from Mr. W. A. Layman stating that it would be impossible for him to serve the Club further. On motion ordered that Mr. Layman's name be withdrawn from the ticket and that of Mr. Julius Baier substituted.

On motion of Mr. Crosby, it was ordered that the Club have its usual annual dinner and that the Executive Committee make the necessary arrangements.

Prof. J. B. Johnson then showed the Club a large number of lantern slides which had been prepared originally to accompany his paper recently read before the St. Louis Railway Club on "The Mechanical Properties of Wrought Iron and Steel, as Shown by Actual Tests," the views being shown this Club by special request. The Professor stated that the paper had already been published in full, including most of the views, and he would be glad to furnish copies to those interested. Adjourned. WILLIAM H. BRYAN, *Secretary.*

445TH MEETING, DECEMBER 16, 1896.—The annual dinner was held at the Southern Hotel. The social meeting in the parlor began at 8 P.M., after which adjournment was had to the small dining-room. Those present sat down to dinner at 8.40. After justice had been done the repast, President Ockerson called the meeting to order, there being forty-three members and seven visitors present. He first announced the result of the election of officers for 1897 as follows: President, Edward Flad; Vice-President, William H. Bryan; Secretary, Richard McCulloch; Treasurer, Thomas B. McMath; Librarian, Julius Baier; Directors, J. A. Ockerson and B. H. Colby; Members Board of Managers of the Association of Engineering Societies, J. B. Johnson and E. A. Hermann.

The Secretary announced an application for membership from A. L. McRae, Consulting Electrical Engineer, endorsed by J. B. Johnson and E. J. Spencer, and G. S. Montgomery, Electrical Engineer with Laclede Power Company, endorsed by William H. Bryan and Abe Cook.

President Ockerson then addressed the Club on the work that it had accomplished during the past year, calling special attention to the excellent condition of its membership list and its finances. In closing, he introduced the new president, Mr. Edward Flad, who presided during the remainder of the evening. Mr. Flad spoke briefly, expressing his thanks for the honor conferred upon him, and pledging the Club his best efforts in advancing its welfare and asking the co-operation of every member in that direction.

President Ockerson then introduced the following resolution which was, on motion, unanimously adopted:

"*Resolved,* That the thanks of the Club be tendered Mr. William H. Bryan for the faithful and efficient manner in which he has discharged every duty devolving upon him as secretary during the past three years."

Mr. Bryan responded briefly, stating that such results as he had accomplished were due as much to the hearty co-operation of individual members as to his own efforts.

Mr. B. H. Colby then responded to the toast, "The Municipal Engineer," explaining the difficulties which beset the pathway of the engineer in city service.

Mr. E. J. Spencer spoke on the "Production and Distribution of Electricity," paying special attention to the advancements which have been made in this science during the past year.

Professor W. S. Chaplin discoursed upon "The Engineer in the Orient," dwelling upon the primitive condition of engineering in the far East, and the limited opportunities for engineers to find employment.

Mr. R. E. McMath addressed the Club on "Civil Service in Municipal Affairs," with special reference to the efforts now being made in this direction before the Charter Revision Commission. He called attention to the good and the weak points of the subject, and made a number of valuable suggestions.

After the completion of the regular programme, Mr. Abbott moved that a committee be appointed to attend the meetings of the Charter Revision Commission, with a view of urging the adoption of an amendment of the charter requiring the president of the Board of Public Improvements to be a civil engineer. After being seconded, the motion was discussed by Messrs. McMath, Pitzman and Holman. Mr. Ockerson moved that, in view of the lateness of the hour, the motion be laid on the table.

Seconded and carried. Adjourned.

William H. Bryan, *Secretary.*

Civil Engineers' Society of St. Paul.

St. Paul, Minn., December 7, 1896.—A regular meeting of the Civil Engineers' Society of St. Paul was held at 8.30 p m.

Present, eleven members and six visitors. Mr. G. L. Wilson in the chair. Minutes of previous meeting read and approved.

A suggestion from the Secretary of the Association of Engineering Societies as to change of letter-head was acted upon favorably, to the extent that the words, "member of the Association of Engineering Societies," be added to the present form. The thanks of the Society were voted Mr. C. F. Loweth for a photograph of the Redwing bridge, built after his plans and under his supervision. County Surveyors William Danforth, of Goodhue County, and C. A. Forbes, of Dakota County, asked the co operation of the Society with the Minnesota Association of Surveyors and Engineers, the Minneapolis Engineers' Club, and the Engineering Department of the State University, in the endeavor to secure legislation in the matter of licensing measurers of land. Mr. G. L. Wilson, Mr. C. F. Loweth and Mr. J. H. Armstrong will serve as a committee in this affair.

Mr. Max Toltz then read a paper on "Paint Tests at the Great Northern Railway Laboratory." The result of the experiments pointed to graphite paints as best adapted for preservation of iron and steel structures. An hour's discussion followed the reading of the paper.

C. L. Annan, *Secretary.*

Montana Society of Civil Engineers.

December 12, 1896.—At the regular meeting of the Montana Society of Civil Engineers, Saturday evening, James S. Keerl, chairman of the Committee of Arrangements for the annual meeting, reported that the preparations were progressing favorably and that the meeting would probably be held in Great Falls. The programme now outlined will include a visit to the Great Falls of the Missouri and the smelters, and a trip to the Belt coal mines, with a banquet at the Park Hotel. It is intended to make the meeting one of the most interesting and in-

structive in the history of the Society, and it is hoped that every member in the State will attend.

The Committee on Nominations named for officers for the ensuing year: Charles W. Goodale, president; A. E. Cummings, first vice-president; M. S. Parker, second vice-president; A. S. Hovey, secretary and librarian; James S. Keerl, member of the Board of Managers of the Association of Engineering Societies; F. W. Blackford, trustee.

William A. Clarke, of Butte, and Frank Leonard, of Libby, were elected to membership in the Society.

The applications for membership of Frank Klepetko and C. W. Sweringer, of Great Falls; Donald Gillies, of Butte, and F. A. Heinze, of Trail, B. C., were read, and the Secretary was instructed to send out the usual letter ballots.

The members present at the meeting were: John Herron, A. E. Cummings, James S. Keerl, F. J. Taylor, Finlay McRae and F. J. Smith.

F. J. Smith, *Secretary.*

The Civil Engineers' Club of Cleveland.

Club Rooms, Case Library, December 10, 1896.—President Howe in the chair. Present, twenty-seven members and twenty visitors.

The minutes of the November meeting, after alterations as suggested by Mr. Searles, were approved.

Messrs. Culley and Osborn were appointed tellers to canvass the ballots for the admission to active membership of Mr. Walter C. Parmley.

The Secretary reported for the Executive Board the acceptance of the resignation of Mr. George E. Gifford, and the suspension, for non-payment of dues, of Mr. H. Grey.

The applications of Messrs. John B. Leeper, Valentine S. Ives, and Edmund M. Sawtelle for active membership, and of Messrs. Charles A. Otis and Edwin S. Mills for associate membership, were read.

It was suggested that the Secretary should hereafter keep a record of how many of those present were members and how many were visitors.

The Secretary offered a motion that hereafter the roll be called at each meeting. The motion was lost, and the Secretary left to find out as best he could. However, the Club passed a motion, introduced by Mr. Searles, that the Visitors' Book be kept open, and that the visitors be requested to sign the same at each meeting under the head of Miscellaneous Business.

Mr. Joseph W. Willard then read the paper of the evening entitled "Explosives: A brief history; their adaptation to the arts and engineering; possible future use in warfare of so-called high explosives."

At 10 p.m., Mr. Willard not having finished, it was agreed to postpone the reading of the remainder of Mr. Willard's paper and the discussion thereof until the next meeting of the Club, and, upon motion of Mr. Dodd, it was voted to have a semi-monthly meeting on the twenty second of this month.

Mr. Parmley was reported as unanimously elected.

The Club then adjourned. After the meeting, a light lunch was served.

F. A. Coburn, *Secretary.*

CASE LIBRARY BUILDING, CLEVELAND, OHIO, December 22, 1896. A semi-monthly meeting of the Club was held in the Club Rooms, Tuesday evening, President Howe in the chair. Present: Twenty-two members and eight visitors.

Mr. Joseph W. Willard presented a short and very interesting paper upon modern "Explosives," their manufacture and mode of use.

Dr. C. F. Mabery, of Case School of Applied Science, followed with a comprehensive demonstration of the chemistry of powder, and the various high explosive compounds, and of their comparative properties and qualities.

After the meeting came the usual interesting visit and lunch.

F. A. COBURN, *Secretary.*

Technical Society of the Pacific Coast.

REGULAR MEETING, DECEMBER 4, 1896.—Called to order at 8.30 P.M. by Past-President Grunsky.

The minutes of the last regular meeting were read and approved.

Mr. Edward F. Haas, Civil Engineer of Berkeley, California, was elected to membership by regular ballot.

The following letter was read:

LONDON, E. C., November 18, 1896.

MR. OTTO VON GELDERN, *Secretary, Technical Society, San Francisco.*

DEAR SIR:—Will you do me the favor to explain to the members of the Society that I very much regret not to have been able to reply at an earlier date to the very kind resolution of sympathy for me, passed by them last May? It was my hope and expectation to have gone to San Francisco last summer for a visit, when I hoped to have had the pleasure of seeing you and to have thanked them in person for their very much esteemed expressions of friendship and sympathy.

My plans at present are so indefinite, that I fear it may be some months before I can indulge myself the pleasure of a trip to America.

Please convey to the members of the Technical Society my cordial good wishes and sincere thanks, and believe me, Yours truly,

(Signed) JOHN HAYES HAMMOND.

A Nominating Committee to select a ticket of officers for the ensuing year was elected by the members present; the five members of the Committee are: C. E. Grunsky, H. C. Behr, Adolph Lietz, Luther Wagoner, and C. I. Hall.

Mr. George W. Dickie read a paper, entitled "Industrial Education," which was discussed by members present.

The importance of the paper read by Mr. Dickie, created a desire to have the subject brought up again for discussion at the next regular meeting of the Society. The Secretary was instructed to prepare an abstract of the paper, and to circulate it to the members to start a more general discussion of the subject of "Industrial Education."

Meeting adjourned. OTTO VON GELDERN, *Secretary.*